Seismic Evaluation Report

for

Building 1003

Onizuka Air Station

1080 Lockheed Way

Sunnyvale, CA 94089

Prepared for

The U.S. Air Force

Onizuka Air Station

Sunnyvale, CA 94089

Contract F04689-96-D-Q001, A & E Services

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Figures

Fig 7. Relationship Of Rock-Soil Accelerations As Shown In Fig 19, Seed And Idriss (Ref. 5)

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ACKNOWLEDGMENTS

We would like to express our appreciation to the staff at the USAF Onizuka Air Station (Patricia D. Thompson, TSgt. Angie Covington, Tony Galam, Marcella Bailey, Bertha Roman and Gene Inserto), for their cooperation and guidance on the administrative procedures, security clearances and photograph permits required during the course of the preparation and review of this report.

EXECUTIVE SUMMARY

MARISCAL ENGINEERING as the prime consultant, and PAL CONSULTANTS INC., are pleased to present the Seismic Evaluation Report for Building 1003 at the USAF Onizuka Air Station (OAS). We are certain that our client, the OAS, will find this report useful in its efforts to comply with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. We recommend that the document be read completely and that we be contacted for any clarification that may be required.

The OAS is a satellite testing and control facility for DOD satellites. The above building provides space for technical personnel and for sensitive equipment performing operations related to satellites, as well as computer rooms and office space for administrative operations and security. The building is classified as an essential facility which requires it to remain undamaged and operational during and after design earthquakes. The building is considered within Risk Group A.

On the basis of our review of available OAS records, as-built drawings, engineering calculations and previous studies, we conducted field evaluations and seismic analyses of this building. Some important soil characteristics and building seismic deficiencies are herein identified.

We have found that building 1003 is susceptible to partial damage from a strong (design) earthquake that may induce limited soil liquefaction as well as affect the steel frame. Building 1003 cannot sustain a strong earthquake without suffering damage due to the steel frame limited capacity and impact effects with the adjacent structures. Nonstructural damage is also expected in this building.

In summary, the building cannot sustain a strong earthquake without experiencing certain damage. Therefore it does not satisfy requirements imposed by the current building occupancy category.

The building will have to undergo seismic rehabilitation if its current building occupancy category is not changed. An alternative to seismic rehabilitation is to downgrade the occupancy category to a building that will experience damage, unlikely to cause collapse, but will not be operational and will need repairs after a major earthquake. In any case, we recommend that OAS adopt measures to eliminate the existing seismic hazards to building occupants.

The rehabilitation work needed to upgrade the building to acceptable seismic safety standards as well as the costs have been estimated. Due to the extent of the work anticipated, we believe OAS will require developing a relocation plan to vacate some offices for a period of construction which is undetermined at this time.

We have achieved the objective of the OAS, which is to evaluate the seismic safety of this building and comply with Presidential Executive Order 12941. The report is accompanied with a tabulated data chart which will be added to the general comprehensive data base that OAS is preparing for its entire facility.

We express our thanks to OAS for the opportunity to provide our services for this evaluation.

1.1 Project Objective, Methodology and Approach

The purpose of this study is to perform the initial Seismic Evaluation of Building 1003 at the USAF Onizuka Air Station (OAS) for potential earthquake-related damage to the building and consequent risk to human life. This building is classified as "Risk Group A." Our task is to present an evaluation report with certain significant facts regarding the physical condition of this building, and the estimated cost of its seismic rehabilitation.

The OAS will incorporate these results into its overall analysis and inventory of OAS buildings in compliance with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. This order is to be implemented as stated in Inventory Screening, Prioritization, and Evaluation of Existing Buildings for Seismic Risk, Engineering Technical Letter ETL 93-3, Air Force Civil Engineering Support Agency, August 1993 (Reference 1).

As stated in our scope of work, our evaluation follows the methods laid out in the NEHRP Handbook for the Seismic Evaluation of Existing Buildings, FEMA 178/June 1992 (Reference 2), which has been modified by Air Force Engineering Technical Letter (ETL) Structural Evaluation of Existing Buildings for Seismic and Wind Loads (Reference 3). These and other codes, technical guidelines and studies are shown in the List of References. Our scope of work is also presented in Appendix A.

Our method of evaluation includes the following steps:

1. Visiting the site and the building to gather data and review pertinent documents of record provided by OAS.

2. Categorizing the building based on its structural type; selecting a set of evaluation statements corresponding to that type; reviewing the statements.

3. Conducting follow-up field work; taking photographs; inspecting critical areas.

4. Performing the analysis required for the evaluation.

5. Performing a final evaluation of the building.

6. Preparing the evaluation report.

We made an initial review of the OAS documents of record to determine the extent to which the existing documents conform to the design and construction standards established for buildings of this kind. A list of these documents is shown in Appendix B.

After reviewing existing drawings and categorizing the building structure according to the FEMA-178/NEHRP, we performed physical inspections in November 1997. Visually evaluating the building, we inspected all areas except those hosting operations that are top security, because authorization to enter these areas could not be granted within the time frame scheduled for the completion of this study.

Using fieldwork questionnaires based on sets of evaluation statements per FEMA-178/NEHRP, we assessed the building elements. We examined accessible construction and compared it with existing

It is important to note that the main structural work in the building was completed about 30 years ago. The building was mostly occupied as offices and computer rooms when we commenced our structural inspection. This meant that we were unable to verify fully the extent to which construction conditions comply with the existing drawings, specifications and codes. Our evaluation of the mechanical and electrical systems is also limited to what is visible and does not include anything concealed in the walls of the building. Our restricted ability to evaluate inaccessible conditions limits our evaluation as well.

Next, we analyzed the building structure and the geotechnical characteristics of the site. The analysis was required for the building elements that we found to be deficient according to the evaluation statements. Since this building belongs to "Risk Group A"-essential facilities that must remain operational during and after an earthquake without posing potential earthquake-related risk to human life, we also gave consideration to nonstructural elements in the building.

During a subsequent evaluation of the building, we determined which elements will need to be seismically rehabilitated. We estimated the cost involved for the rehabilitation work and tabulated the results in a chart to be included with the comprehensive analysis and inventory of OAS buildings.

1.2 Report Organization

The report starts with an Executive Summary that gives the essential conclusions of our evaluation. The body of the report is presented in 10 sections. Section 1-Introduction--is presented herein. Section 2-Building Location and Description-offers a summarized description of the OAS site and gives the location of the building.

Section 3-Documents of Record-covers our review of existing information on the buildings. Section 4-Geotechnical, Site Geology and Soils-offers our evaluation of the site seismicity based on the available studies at OAS.

Sections 5 provides information about Building 1003. In this section we describe the circumstances under which we gathered field data; we provide our evaluation of the building; we show the seismic analysis criteria we used and our results; and we give a list of structural deficiencies for the building.

Section 6-Building Deficiency Mitigation and Cost of Seismic Rehabilitation-presents the list of deficiencies to be mitigated as well as the estimated cost for the seismic rehabilitation of Building 1003. Section 7-Conclusions and Recommendations-includes our best judgment and answers to the problem issues of Building 1003.

We have attached five appendices to the Report. Appendix A-Scope of Work-describes a prioritized work scope issued by OAS. Appendix B-Reviewed Documents of Record-lists the documents provided to us for review.

Appendix C-Photographs, Evaluation, Seismic Analysis, Deficiencies and Cost-show photographs; our findings during the physical inspection of Building 1003; the seismic analysis; a list of found deficiencies; and costs for the seismic rehabilitation of the building.

Appendix D-Data Base-shows the tabulated data base of the evaluation results for Building 1003 presented in the required format. Appendix E-Project Directory-provides information on the individuals participating in the project.

SECTION 2. BUILDING LOCATION AND DESCRIPTION

The OAS is a satellite testing and control facility located on a 22-acre campus along Mathilda Avenue and Moffett Park Drive in Sunnyvale, California. Both streets provide the boundaries for the air station. The site is a broad, flat area, adjacent to the Lockheed Martin Company, southeast of the Moffett Airfield. The facility consists of several buildings as seen in Figure No. 1.

Maps dating back to 1959 show this parcel of land mostly vacant, with the Research and Development Building and a gas plant as the only structures on the site. Starting in 1959, a mix of various building types were built to create the OAS and provide satellite telecommunication testing and control services to the USAF. This mix of buildings is still present, together with more recent additions, such as office trailers and other semipermanent structures.

Figure 2 shows an artistic view of the site with Building 1003 at the center. This figure was obtained from a rendering of the OAS facilities that is exhibited in the main lobby of Building 1001. Some pictures of the building and its surroundings are also shown in Appendix C.

Building 1003 is located in the central part of the OAS. The building is a five-story, rectangular shape, 170,400-square-foot office building. It provides necessary offices for staff, equipment, computer rooms, technical, and administrative operations. It was originally built in 1968. The structure is primarily steel-frame, with precast concrete panels along the perimeter walls. The foundation is a slab-on-grade supported by concrete piles.

The first floor is a reinforced concrete slab on grade. The second through the fifth floors are concrete/metal deck and raised tile floors with space for computer and telecommunication equipment wiring. The roof is a flat, built-up roof over insulation and metal deck.

Our visit to the site gave us an immediate impression about the level of closeness between Building 1003 and the adjacent buildings. This proximity creates important issues, as it affects emergency evacuation routes in the case of an earthquake.

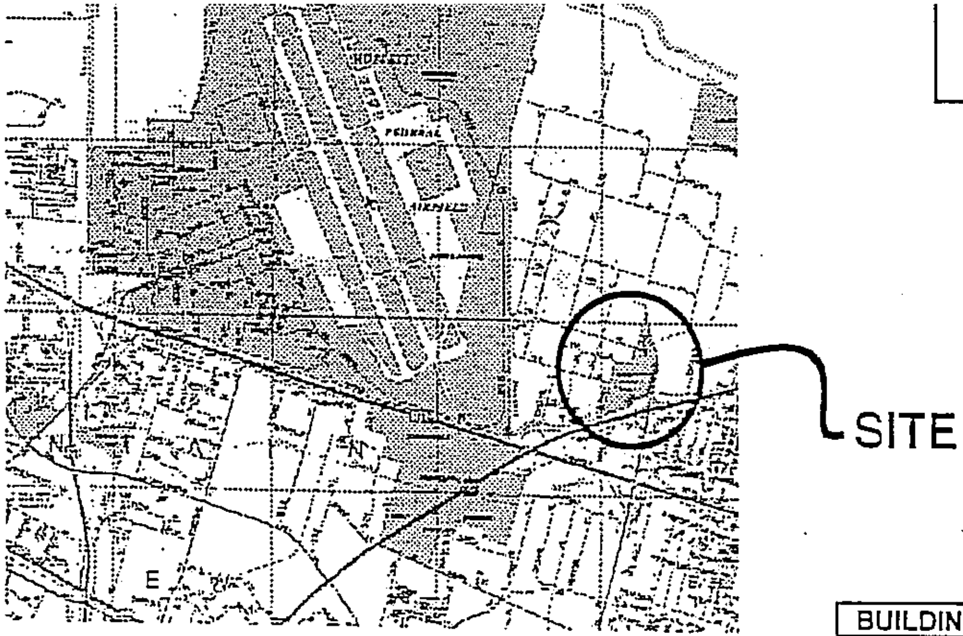
Other "semipermanent" structures have been added in the rear vicinity. Between Building 1001 and Building 1003 there is an open area which has been altered to include a steel deck structure for a gym and a volleyball court.

Buildings 10031 and 10032 were added to the north and west of Building 1003 respectively.

The area in between Buildings 1001 and 10031 has been utilized as a covered passage to provide access to Building 1003. This addition of surrounding structures contributes to the feeling of enclosure.

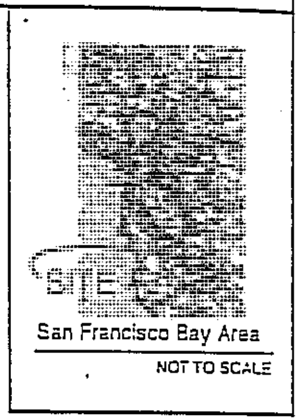
There are additional metal structures on the roof of Building 1003 that house HVAC equipment upgrades and minor electrical and mechanical piping. Various antennas and telecommunication improvements have also been added on top of this roof.

The main entrance of Building 1003 is through a lobby located at the northwest corner of Building 1001. Potential zones for evacuating the buildings in an emergency are the landscaped frontyard near Building 1002 and the paved parking areas behind the building.

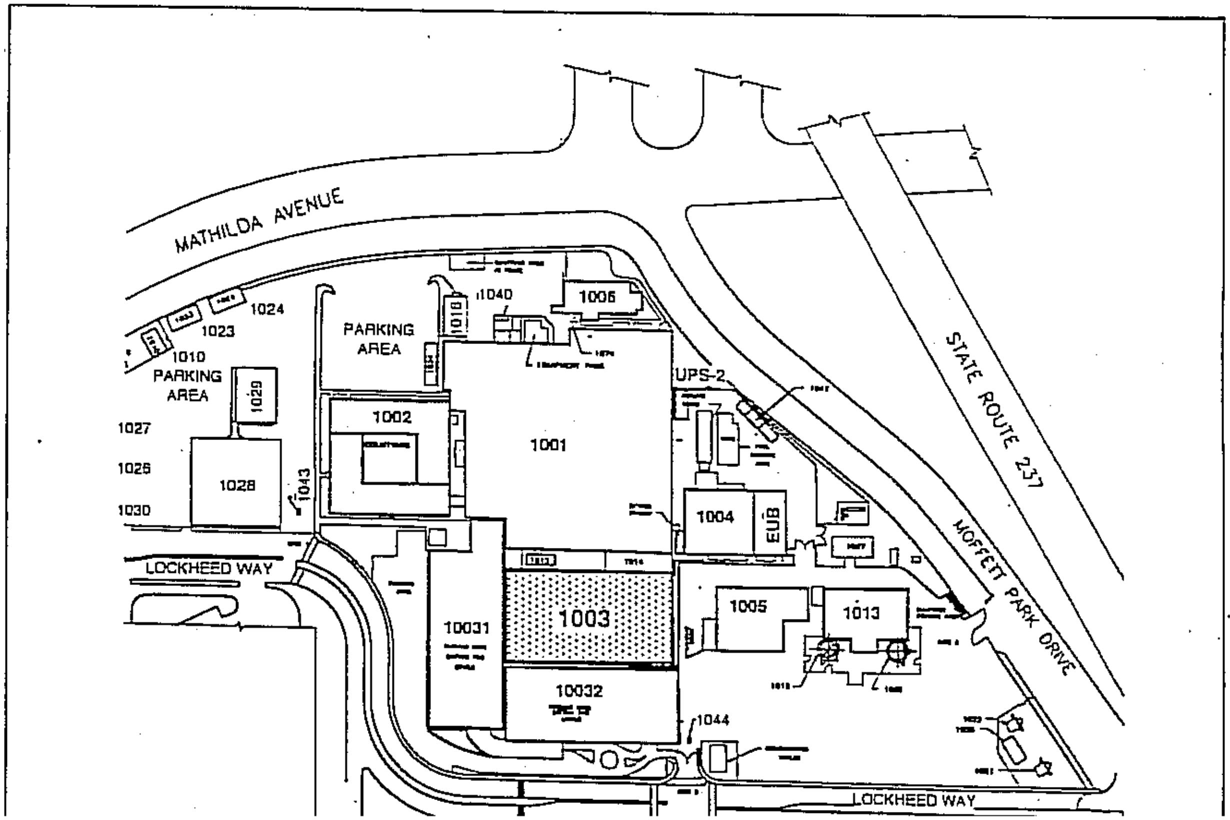


SUNNYVALE, MOFFETT / ONIZUKA VICINITY MAP

NOT TO SCALE



BUILDING	AREA (S.F.)	AREA (M2)
1003	170.000	15.793



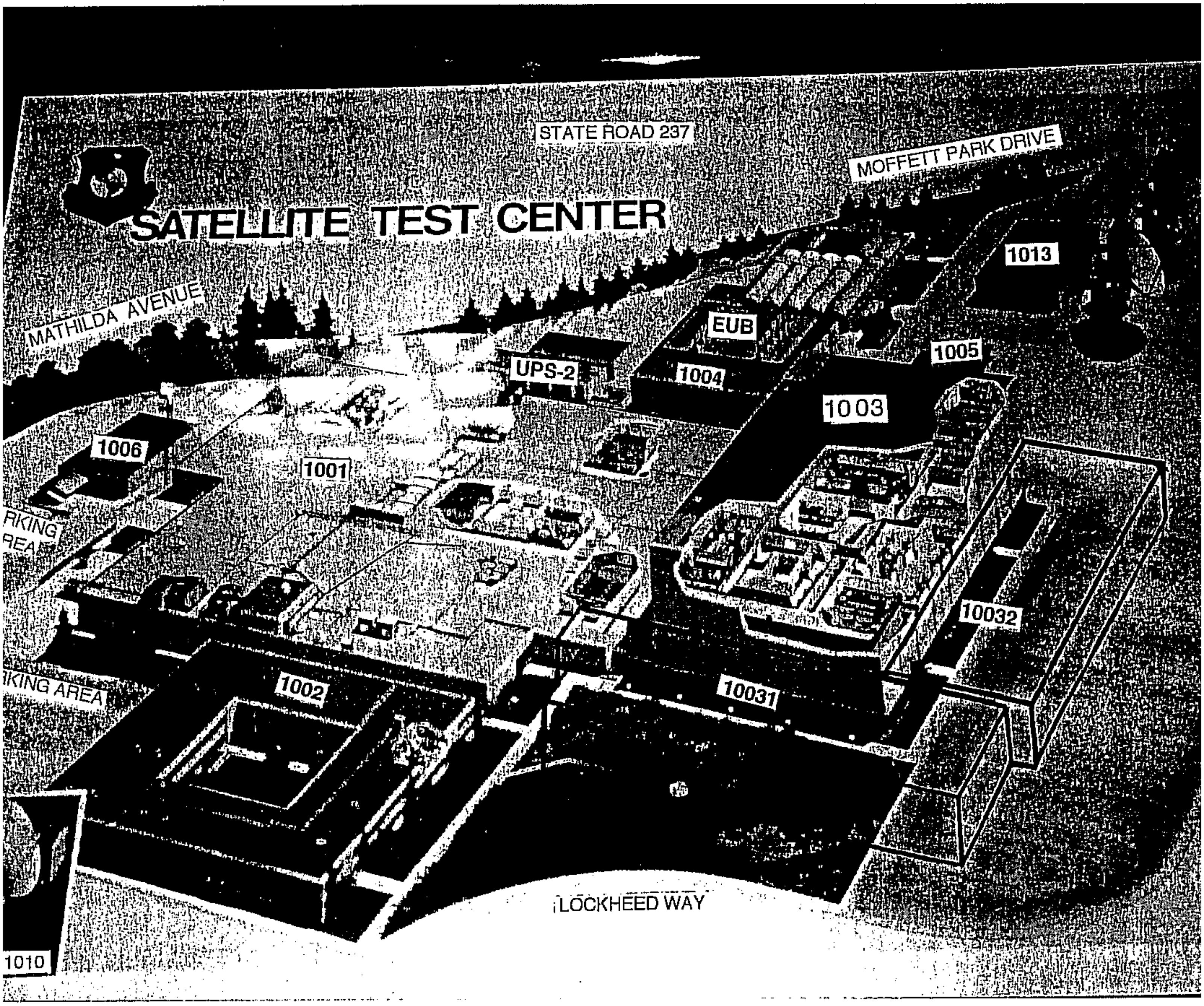


FIG 2. ONIZUKA AIR STATION, SUNNYVALE, CA.

During our initial visit to the site, we discussed with OAS staff our intention to assemble and review as much of the existing information on the Building 1003 as possible. We requested building design data, including original construction drawings, specifications, and calculations. We also requested geotechnical reports of studies performed at the site for the design and construction of Building 1003.

We intended to review and make use of existing analyses to assess the basis of the earlier work on Building 1003. Any similar information on remodeling work or other data, such as assessments of the building's performance following past earthquakes, was also requested. We also wanted to evaluate the site to identify geologic hazards to the building.

A summary of the data and documents of record from the OAS that were given to us for review is shown in Appendix B. These documents include the as-built construction drawings, structural calculations and a set of existing geotechnical reports dating from 1959 to 1993 on site soils conditions and soil boring locations. Preliminary seismic studies for Building 1003 were also available for our review. These studies contained recommendations for the seismic retrofitting of the building.

Efforts to locate information on the actual construction of seismic repairs were unsuccessful. Furthermore, OAS has no records of any methodical program for post-earthquake assessments of seismic performance for Building 1003.

We also inquired if OAS had recent data on underground water table levels from any existing monitoring well at the site. In addition, we wanted to know if there was any information, such as seismographic data from the 1989 Loma Prieta Earthquake, recorded at the site. This major earthquake occurred about 30 miles south of the site. No such data seems to exist and there is no additional information on the current seismic status of most OAS buildings.

The intent of the initial document review phase was to determine the extent to which the actual construction of Building 1003 conforms to the existing documents.

In addition, we looked at the record maps and soils studies for the area and this gave us an idea of the building structural designs that took place at the site in the last 30 years. Based on our review of this geotechnical information, and recent developments in the geotechnical sciences, we identified soils conditions that present moderate risk at the site.

Comments and conclusions from our review of existing data are given throughout the following sections.

According to the scope of work, we evaluated the OAS site for geological hazards based on FEMA-178/NEHRP Handbook for the Seismic Evaluation of Existing Buildings (Reference 2). Our intent was to make a minimal assessment of the site to identify geologic hazards that might affect Building 1003, based on geotechnical characteristics shown in the existing studies. No soil borings or other underground soil samplings were authorized for this study.

4.1 General

We reviewed the conditions for this site based on the collection of soils studies and test borings in the vicinity, which were conducted from 1959 through 1993. These studies are listed in Appendix B. The same studies have also been used in a previous evaluation of other buildings at OAS (Reference 4 and 8).

A further inquiry to the OAS revealed that valuable information such as local seismographic recorded data from the 1989 Loma Prieta Earthquake or recent data on underground water table levels at the site do not exist.

The OAS site is located in the San Francisco Bay Area, a seismically active zone (FEMA-178/NEHRP Zone 7 and Uniform Building Code Zone 4), with large, active faults. As shown in geological maps, the OAS site is located approximately 6 miles southwest of the Hayward Fault, 12 miles southwest of the Calaveras Fault, and 10 miles northeast of the San Andreas Fault.

On the basis of state-of-the-art knowledge, we can say that no known active fault, capable of surface rupture, has been reported across the OAS site. There are no visible signs of displacement or rupture. The OAS is in a flat, low-lying area, next to the marsh lands south of the San Francisco Bay. There are no slopes for several miles around, so the site is protected from the risk of potential earthquake-induced slope failures or rockfalls. As for the risk of "tsunamis" or "seiches", investigation in this area for the region surrounding the San Francisco Bay is in its preliminary research phase, with no practical conclusions or applications.

In 1993, a Final Report for Seismic Design Criteria for Building 1003 was issued by Harding Lawson Associates (Reference 4). This report assumes three probabilistic earthquakes EQ-I, EQ-II and DE respectively. EQ-I has a 50 percent probability of being exceeded in 50 years. EQ-II has a 10 percent probability of being exceeded in 100 years. DE has a 10 percent probability of being exceeded in 50 years.

Moderate earthquakes with maximum credible magnitude of 6 within 20-kilometer and 100-kilometer radiuses of the site were used to assess the seismic risk at the site. We believe that a higher magnitude such as 7.5 or 8 should have been used to analyze seismic risk at this site. Also, it is not clear-in this or in any other of the existing reports-whether any site specific investigation of faults or lineaments were made or not.

From reviewing geological maps, we concurred with the site description given in the existing reports. The site is located on a flat alluvial valley within the area known as the California Coast Range geologic province. The soil formation here consists of a series of northwest-bearing mountain ranges underlain by faulted and folded rocks. There are large, active faults in this range.

... and sand and gravel layers.

The existing reports also show that the groundwater table appears to be located between 11 and 16 feet below street level. The groundwater data is very critical for the evaluation of soil liquefaction potential, but often the groundwater level recorded during drilling in highly clay soils is misleading. Also, ground water level could vary due to seasonal factors such as prolonged dry or wet periods.

From information on the existing borings, we produced a soil boring plot plan and few soil cross section profiles across the site which are shown in Figures 3, 4, and 5 and 6. Based on the soil profile and on FEMA-178/NEHRP, the soil profile type can be classified as S2 type. The site coefficient is S = 1.2 for this soil type.

After review of the available data, we arrived at the following conclusions.

4.2 Design Response Spectra

The 1993 study (Reference 4) by Harding Lawson Associates (HLA) provided design response spectra for three levels of design motion parameters corresponding to probabilistic earthquakes EQ-I, EQ-II and DE. See Figure 4 of such study.

While we do not question the theory behind HLA's seismic risk analysis, we are surprised at their results. The shape of the design response spectra shows that the natural ground period would be:

T = within 0.2 and 0.3 seconds, which is typical of rocky or very stiff sites.

Given HLA's characterization of the area as an alluvial site with abundant silts and sands, test blowcounts in the 30's, 20's, 10's, and occasionally lower, a thickness between 500 and 600 feet, and ground water levels between 11 and 16 feet, this type of profile would perhaps rather correspond to the so-called "Deep Cohesionless Soil" as defined in Ground Motions and Soil Liquefaction During Earthquakes, Seed & Idriss, 1982, Earthquake Engineering Research Institute (Reference 5).

On the other hand, our experience with other soil profiles on alluvial sites next to the bay near the western end of the San Mateo-Hayward Bridge north of the OAS site, shows typical ground periods between 0.6 and 1.2 seconds.

We would therefore expect the "true" ground period at OAS to be somewhere between these two scenarios.

We recommend an analysis of the fundamental period of ground shaking as a verification of the HLA's period. This could be accomplished either experimentally in the field, by performing a geophysical survey, and/or analytically, by analyzing the ground as a multistory shear structure using computer modeling procedures.

We also recommend, in a future Building Seismic Rehabilitation Program at OAS, that more soils test borings be performed, and that ground motions of surficial soil layers be determined by analyzing the vertical propagation of rock motions to the surface. This could be achieved through a more accurate soil characterization based on additional new borings and state-of-the-art methods of calculation.

earthquakes. The liquefaction potential remains low for the EQ-I earthquake, except at the locations where the N values are below 20. However, if our proposed value of A max = 0.19g is used for the EQ-I earthquake, the liquefaction potential becomes marginal.

Nonetheless, we are still concerned about liquefaction occurring in the silty sand layers that have a N value equal to 7. This value is found first in a 2.5- to 4-foot-thick layer at a 10- to 15-foot depth, and also in a 4- to 6-foot-thick layer at a 30- to 35-feet depth. A compiled soil profile, based on the "Old" and "New" borings, shows a localized "problem zone" located under Building 1001. The Seed & Idriss method yields a high liquefaction potential in these layers for all three earthquakes.

However, further study is warranted in these cases to determine-among other things-the percentage of fines, including clay contents, which may vary the soil vulnerability to liquefaction.

We again recommend that, prior to the start of a comprehensive Building Seismic Rehabilitation Program at OAS, additional soils test borings be performed to obtain more accurate soil profiles.

If an updated study based on new data confirms that the liquefaction potential still exists, possible avenues of mitigation could include (among other methods) additional and deeper building foundations or soil densification by grout injection.

4.4 Seismic Stability of the Building Foundation Design

The most recent borings located in the close proximity of Building 1003 are described in two separate reports (References 4b and 4c). Using the data contained in these reports, we conclude as follows:

a. The site of Onizuka Air Force Base is underlain by over seven hundred feet of predominantly blue, gray and green clays formed during periods of aqueous deposition, including marine clays (Old Bay Muds). These clays may also include layers of yellow and brown oxidized clays of continental deposition. The upper 10 to 15 feet of natural subsurface materials are probably recent alluvial fan deposits. The clay layers contain varying thickness of coarse grained channel or stream deposits, such as loose to medium dense silty and sandy layers 8 to 10 feet in thickness. These deposits, which are moist to wet, consist predominantly of clayey silts and clayey sands, with lenses of loose sands at and below the groundwater table.

b. The groundwater table occurs between 11 to 17 feet below the existing ground surface. Fluctuations in the groundwater level should be expected due to seasonal changes, variations in rainfall, and other factors.

c. The drilled cast in place piers (2 to 3 feet diameter, 45 to 60 feet deep) supporting Building 1003 are primarily skin friction type, because only a small amount of end-bearing will be developed owing to the clayey nature of the in-situ soils at the bottom of the drilled piers. These piers are designed with a minimum factor of safety of two.

d. Based on the standard penetration test data of the loose sand deposits, it appears that these would liquefy in a major earthquake. Nonetheless, even if the loose, 10 to 15 feet thick, sandy strata liquefy during postulated major earthquakes (probability of exceedance greater than 50-year, EQ1 or 100-year interval, EQ2), the loss of skin frictional vertical and horizontal capacities of these sixty feet deep piers are not expected to degrade by such large amount as to result in collapse of the Building structure. Nonetheless, a pier reinforcing system is recommended.

With an initial visit to the site, a review of the record documents, and using the FEMA-178/NEHRP Handbook for the Seismic Evaluation of Existing Buildings (Reference 2), we classified Building 1003 structurally and selected a set of evaluation statements corresponding to the building type. We utilized these statements, which come in questionnaire format, during our field work.

After categorizing the building structure, we scheduled a physical inspection phase so that we could visually inspect the building in November 1997. Our access to the building was restricted for security reasons and advanced notice to the OAS Base Civil Engineer was required in order to arrange for inspections and security escorts.

We were authorized to inspect most of the building, except for the central core which is not available for inspection due to high security restrictions.

We examined accessible construction and compared it with existing documents, but our field work and the scope of our inspection was limited and did not include any destructive tests, hole punching, or any kind of rupture test. Any potential, latent and inaccessible defects are, therefore, excluded from this report.

It is important to note that all structural work in the building was long ago completed. Also, the building offices were mostly occupied when we commenced our inspection. As a result, we were unable to fully verify the extent to which construction conditions comply with the existing drawings.

Our evaluation of the mechanical and electrical systems is also limited to what is visible and not concealed in the walls of the building. In addition, our restricted ability to evaluate inaccessible conditions limits the evaluation. In future seismic rehabilitation plans, destructive and nondestructive testing of some elements may be necessary to determine capacity and quality. A limited amount of exposure of critical connections and reinforcement may have to be made to verify conformance to the existing drawings.

It is important to note that the structure is exposed on the mezzanine floor of this building which facilitated our inspection. Unfurred walls provided us with an unobstructed view of the steel structural components. As a result, we were able to verify the extent to which construction conditions comply with the existing drawings at this location.

During this phase, we took photographs to the extent allowed by security restrictions and we used the questionnaires (with evaluation statements corresponding to the building type) shown in Appendix C.

In these questionnaires, if a statement is found to be true, the condition being evaluated is acceptable and the issue may be set aside. If a statement is found to be false, it means that a condition exists that needs to be addressed further, since it may lead to a serious seismic deficiency.

5.1 Structural System

Building 1003 was constructed in 1967. A rectangular-shaped building with a flat roof, four-story high, with a mezzanine between the second and third floors, it measures 143 feet by 258 feet, with a gross area of approximately 170,000 square feet. It has large ceiling spaces between floors for electrical and mechanical ducts.

stories with 19 to 25 feet story heights. The first floor throughout the building is a reinforced concrete slab on-grade. The second throughout the fourth floors are concrete/metal deck with raised tile floor areas. There is a partial Mezzanine floor between the second and third floors. The foundations are grade beams and deep piles.

The lateral-force-resisting system therefore is the bracing system. Lateral loads are transferred by diaphragm to braced frames. The roof and floors are expected to act as the diaphragms. The vertical components of the lateral-force-resisting system are the braced frames.

We found that the building evaluation involved several substantial difficulties. One was the fact that the structure is hidden by architectural finishes. On the outside the structure is concealed by exterior curtain walls (precast concrete panels), while on the inside it is covered by column furring and ceilings. Access into ceiling space was also difficult. Some rooms, however, like the mezzanine and bathrooms, allowed us views of the structural elements and ceiling space in adjacent areas.

The perimeter curtain walls have few door openings and no windows. From the exterior, the curtain walls look in good condition. We carefully inspected them along the base floor and at the corners where shear stresses usually produce failure, but we found no cracks. We also inspected some of the steel brackets that serve to attach the interior face of the curtain wall precast panels onto the steel building frame.

The roof framing system appears to be in good condition although the tack-weld connection of the metal deck to the steel joists is inadequate for a full diaphragm effect. Climbing the metal staircase in the fourth floor that provides access to the roof, we examined and found the built-up roofing surface in good condition. We attempted to find signs of roof leaks that could be causing corrosion of the roof structure, but the maintenance work is good and we saw no signs that the roof is leaking or in need of repair or replacement.

Steel braced-frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large interstory drifts that may lead to extensive nonstructural damage. Also, because of the irregular location of the mezzanine, the west part of the building is more flexible than the east part. This could result in torsional displacements that might cause damage to nonstructural elements.

The structure lacks an adequate lateral-force-resisting system. The diagonal braces and foundations are overstressed. There are no in-plane braces in the floor and roof slabs. Damage was observed in 32 connections following the 1989 Loma Prieta earthquake. Some repairs were made in 1992 as described in a later section.

5.2 Nonstructural Systems

Investigation of nonstructural elements was very time consuming because these elements are not well detailed on the plans and most are concealed. It was essential, however, for us to investigate these items since nonstructural elements can pose significant hazards to life safety under certain circumstances. Our concerns had to do with:

* Seismically induced forces acting directly on the nonstructural elements.

* Interaction of the structural system with the nonstructural element as a result of the nonstructural clement becoming load bearing due to lack of separation.

We inspected nonstructural elements to address their overall conceptual seismic status. We were concerned, during our inspection, that their seismic support might have been given little attention in the past during alterations to the building, making them potentially dangerous.

In addition, we learned that there are building contents that pose hazards because items such as batteries, toxic chemicals, oxygen tanks, and flammable substances are stored in some rooms. The potential harm of these materials also warranted our attention during this phase of the evaluation, and we inspected storage conditions as well as supports, restraints, clamps and other means of preventing the overturning of containers and spilling of these materials.

We also stressed life-safety objectives having to do with evacuation and rescue of building occupants during an earthquake.

The nonstructural elements that have a possible life-safety hazard are identified in the list of deficiencies.

5.2.1 Partition Walls

The 'nonstructural partition walls' are those interior walls that are not part of the seismic load carrying system. To ensure their nonload-bearing condition, we focused on their attachment and interaction with other elements and checked if these conditions had been altered without seismic design consideration.

The building does not have unbraced, unreinforced masonry, or hollow clay tiles that are brittle. The partition walls are made of metal studs with gypsum board and have some rigidity. In some places, they are connected at top and bottom to the steel frame columns. They will participate in resisting lateral forces in proportion to their rigidity relative to other building systems. And they will take a minor portion of the lateral load at low force levels. At some higher level, however, they will crack and lose strength before the main system takes all the lateral load.

We found at structural separations that the partition walls did not always have seismic or control joints. These joints are not provided at perimeter cross walls, core walls, and long walls. We also noticed that the tops of partitions that only extend to the ceiling line did not always have lateral bracing to prevent overturning or buckling.

5.2.2 Ceiling Systems

Ceilings in the corridors and offices consist of suspended T-bar rails and lay-in tiles. Ceilings at only a few locations along the corridors are suspended gypsum board attached to ceiling joists.

LS Neither the suspended ceiling or the ceiling-supported lighting and mechanical fixtures are adequately braced. Consequently, these ceilings may have problems during an earthquake. The size and shape or the continuity of light fixtures may also affect the performance of the ceiling element.

LS The tile ceiling system weighs very little and will require both compression members for lateral/vertical bracing in addition to the tension wire supports for vertical weight. These supports will be needed to prevent lay-in boards from jolting and dropping out of the grid. Clips will also have to be installed to improve the performance in areas that people will be using to exit the building.

... placed, the ceiling system extends continuously across seismic joints or building separations. Seismic or control joints will have to be provided at structural separations, perimeters, structure penetrations, and core walls, and in areas where the ceiling configuration indicates that a torsional condition may occur.

5.2.3 Electrical System and Light Fixtures

LS The building has a lay-in fluorescent lighting system. We found that the light fixtures are not always supported and braced independently of the ceiling suspension system, which means any ceiling movement could cause the fixtures to separate and fall from the suspension systems. These fixtures will have to be supported separately from the ceiling system or be provided with a backup support that is independent of the ceiling system.

LS The diffusers on the fluorescent light fixtures are not supplied with safety devices or some other form of positive attachment.

LS We also found stem-hung incandescent systems that had fixtures suspended from stems or chains. The swinging of these fixtures could cause the light and/or the fixture to break after striking other building components. Also, the stem connection to structural elements could fail. Fixtures might twist severely, causing breakage in stems or chains. Long rows of fixtures placed end to end could be damaged due to this kind of interaction. Long-stem fixtures will tend to suffer more damage than short-stem units.

LS In other parts of the building, we found surface-mounted incandescent systems. The ceiling-mounted fixtures can separate and fall from their suspension systems during ceiling movement. The wall-mounted fixtures are well attached and will perform well seismically.

LS We also noticed some surface-mounted fluorescent systems on ceilings and walls. Ceiling-mounted fixtures will perform in a fashion similar to lay-in fixtures, while wall fixtures will perform better than ceiling fixtures. However, parts within the fixture could separate from the housing and fall.

LS We also saw a few pendant light fixtures and double-stem fluorescent fixtures that will need better lateral supports. These fixtures without lateral bracing are located at the mezzanine floor.

All the emergency lighting equipment and signs are anchored and/or braced to resist vertical and horizontal earthquake loads.

5.2.4 Cladding, Glazing and Veneer

All exterior wall cladding consists of curtain walls made of precast concrete panels which are properly anchored to exterior wall steel framing for in-plane and out-of-plane lateral forces. Connections to the building frames have sufficient strength and/or ductility to prevent exterior wall panels from falling. Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts.

There are at least four connections for each wall panel that are capable of resisting out-of-plane forces. Where bearing connections are required, there are at least two bearing connections for each wall panel.

As we could observe from the ground level up, there is no cracking in the panel materials that may be indicative of substantial structural distress. We checked exterior walls for deterioration, but we did not find cracking on the exterior walls due to thermal movements.

The wall panel joints are covered with neoprene joints. These joints as observed from the interior at the mezzanine floor seem to be in good condition and do not show traces of water leakage.

The building has no exterior windows.

LS We gave special attention to the glass/wall at the lobby because of its use as an entrance and exit way. The partitions and fixed glass at the lobby are not detailed to accommodate the expected frame drift. Glazing is not isolated to accept predicted drift without shattering. Although the glass frame is in good condition, we did notice that the glass in these frames is another element that could stiffen the frame if the frame drift exceeds the amount of slip between the glass and its frame. For safety, the glass could be replaced with stronger, tempered or wire glass set in a frame that would allow for in-plane movement.

5.2.5 Parapets, Cornices, Ornamentation, and Appendages

LS The building has parapets above the roof which are extensions of the wall panels. As shown on existing drawings, these concrete parapets, up to 5 feet high, have vertical reinforcement but no diagonal bracing.

There are no laterally unsupported unreinforced masonry parapets or cornices in this building. Other appendages, such as vents that extend above the highest anchorage level of the roof, are braced and well anchored to the structural system.

The cornices that cantilever from the exterior wall faces are reinforced and well anchored to the structural system.

The building has no signs, chimney or other appendages that could represent hazards. The rainwater downspouts, drains and drain pipes are also well attached.

5.2.6 Means of Egress

Building 1003 has no walls made of hollow clay tile or unreinforced masonry which could fail and litter stairs and corridors.

The building has no proper setbacks to separate it from the adjoining buildings along the sides and rear. However, there is a covered passage that leads to a lobby in Building 1001 and to an open area.

In all the floors above, the hallways, located in square configurations around the central core, conform to current requirements for emergency exiting and lead to the staircases.

Corridor doors are properly framed and should not jam due to partition distortion.

Cornices, canopies, and other ornamentation above building exits are well anchored to the structural system. Canopies are anchored and braced to prevent collapse and blockage or building exits. We do not expect these elements to fall and block egress. There are no hanging signs or anything in the roofing that is within a distance of 10 feet on either side of an opening or in any place where an occupant can walk.

LS Lay-in ceiling boards and tiles used in exits or corridors are not always secured with clips. This should be done to prevent tiles from falling and hindering egress in high occupancy situations. Lay-in ceiling boards

5.2.7 Staircases, Elevators and Freight/People-Moving Equipment

The staircases are located at each building corner. Staircases are built of steel framing which allows for drift and ensures that it will not be seismically interactive with the structural system. We do not expect it to fail. The steel railing bars of the staircase are properly painted over, and we found no signs of moisture-induced rust or corrosion of exposed steel that could affect the structural strength of the staircase steel members. We are concerned, however, about the fact that some of the railing is not well anchored and produce excessive vibration. The roof is accessed by a metal staircase in the fourth floor.

There are steel catwalks on the roof for access to equipment. These catwalks are well maintained and serve to cross over pipeways, equipment and other obstructions. They need some minor bracing.

The building has passenger and service elevators which are in good condition. There are no escalators, hoists or any other freight/people-moving system.

5.2.8 Electrical, Mechanical and Miscellaneous Equipment

The electrical service is fed to a main panel and split out to breakers for the site service and the individual offices. Power is then distributed to sub-panels located at various parts of the building. Due to site security, no access was provided to inspect the main panel, but we saw various subpanels in properly attached conditions.

Because the electrical service will have to remain operational after an earthquake, we assume there is a back-up emergency power supply to the building. Any lack of power or failure of circuit breakers or wiring could be detrimental to OAS operations.

The equipment for the heating and air conditioning system and other exhaust systems, chillers, and ventilating fans, are mounted on special concrete or steel decks built at the mezzanine floor and roof top. Some of the equipment is housed in utility rooms. These structures provide adequate coverage to the equipment.

Various antennas and telecommunication equipment are located on the roof structure. Their supports are in good condition but additional lateral bracing is needed at a few places. There is no additional heavy loading on the roof.

We observed the elevator equipment room on the fourth floor of the building. The system is operating without excessive vibration, leakage, or noticeable maintenance deficiencies.

We also saw equipment in the mezzanine floor. Equipment such as chillers, tanks, generators, fans and pumps are mounted on concrete pads and anchored with steel connectors. Some connectors are vibration isolators equipped with restraints or snubbers to limit horizontal and vertical motion. However other supports are rigidly connected to the pads and are causing cracking problems. Also, shearing of anchor bolts can occur on rigidly mounted large equipment and lead to horizontal motion. Once unanchored, equipment may move and damage utility connections and parts of the roof.

No pieces of major mechanical equipment are suspended from the structure without seismic bracing. We found most of the mechanical and electrical equipment adequately anchored to the structure or foundation.

In terms of life-safety concerns, we found that the mechanical and electrical evacuation equipment is properly mounted and should still be operable after an earthquake.

At non-inspected security areas and areas being currently remodeled, we recommend that all equipment supported on access floor systems should either be directly attached to the structure or be fastened to a laterally braced floor system.

The equipment maintenance schedule appears regular and diligent. Future equipment additions or replacement needs should be considered with the possible effects of an earthquake on the building structure in mind. To reduce unnecessary additional weight to the building floors or roof, equipment which is no longer in use should be dismantled and removed.

5.2.9 Piping, Ducts and Utilities

Our examination of fire suppression piping, including sprinkler system piping and standpipes, found the risers anchored and braced with flexible couplings that will allow for building drift and floor movement. We expect the system to be operational after a seismic occurrence.

We also found a great amount of insulated chilled water and steam piping at the mezzanine floor. Some of the piping is coming from outside the building and onto the mezzanine through unsealed openings in the firewall. This piping is anchored and braced to prevent failure at elbows, tees and at connections to supported equipment. Not many flexible joints are provided, however, and the potential for piping failure is dependent upon the rigidity, ductility, and expansion or movement capability of the piping system. Joints may separate, and hangers may fail; hanger failures in turn can cause progressive failure of other hangers or supports.

The greater flexibility of the small diameter pipes will allow them to perform better than larger diameter pipes, but they are still subject to damage at the joints. Although gas piping less than 1 inch in diameter and other piping less than 2-1/2 inches in diameter need not be braced, we recommend that minimum bracing be installed.

The insulated piping alignment traverses along various adjacent buildings. The piping layout contains various expansion loops with restraints in between. A future check of the piping system is recommended. Shutoff devices provided at building utility interfaces to stop the flow or gas, high temperature steam, etc., in the event of earthquake-induced failure should be tested periodically.

We noticed that some pipes cross building separations without a flexible connector. Failures may occur in these pipes due to differential movements and adjacent rigid supports. We also noticed some places where upgrades are needed. Examples are pipes that are supported by other pipes and some major piping supported by unrestrained one-side C-clamps.

All the pipe sleeve wall openings have a diameter of less than 2 inches larger than the pipe. However, special consideration must be given to the sealing of penetrations of firewalls, fire-rated assemblies, and smoke-stop partitions.

structural elements.

Duct work in long lines is laterally braced along its entire length but is also in need of some upgrade. Failures may occur in long runs due to large amplitude swaying, though failure usually results only in leakage and not in collapse. Some ducts do have flexible sections in places where they cross seismic joints.

The main domestic water supply line that runs throughout the building is well supported. Plumbing for bathroom toilets, sinks and accessories is in good condition. The sanitary sewer lines that drain bathroom fixtures and the venting lines up to the roof are in adequate condition. The building roof drains seem to be in good condition.

After an earthquake, flush tests should be performed for maintenance and repairs. Rupture and clogging of the sanitary sewer and the storm drain line could cause backups that might damage floor-mounted, moisture-sensitive equipment.

Additionally, there are no lighting, telephone or any other aerial cables coming to the building from lightposts that could fail during an earthquake.

5.2.10 Telephone, Signal and Security

Although due to site security, we were not permitted to inspect the telephone, signal and security systems, we assume that these services run to a main telephone panel which is well anchored and supported.

Also, the subpanels and fixtures of these systems need to be inspected to ensure that they are properly anchored. Elements of the fire alarm system-such as the site fire alarm pull station or call box, as well as the emergency telephones, security alarm system, security-activated doors and gates-all must remain operational and connected with OAS's central security system after an earthquake.

5.2.11 Environmental, Health, Safety and Hazardous Materials

Asbestos-containing materials or similar building materials that may experience an unnoticeable release of particles during an earthquake were not part of our scope of work. A full environmental site assessment is available at OAS.

We limited our work to inspections of areas of the building where hazardous materials are stored.

We focused on materials such as compressed gas cylinders, chemicals and other flammable materials. Because of the secondary dangers that can result from damage to vessels containing these hazardous materials, we checked to see if they were properly braced and restrained.

We found that all compressed gas cylinders are restrained against motion, thus forestalling the release and ignition of fumes.

Piping containing hazardous materials is provided with shut-off valves or other devices to prevent major spills or leaks.

Our inspection revealed that most of the tall, narrow storage racks, bookcases, file cabinets, or similar heavy items are anchored to the floor slab or adjacent partition walls. This was done to prevent them from tipping over. File cabinets arranged in groups are also attached to one another to increase their stability.

LS Most cabinet drawers have latches to keep them closed during shaking. A few unlatched vertical cabinet drawers need to be secured or they may swing open, allowing their stored contents to fall out. This could be a problem with cabinets located adjacent to exit routes.

Breakable items stored on shelves are restrained from falling by latched doors, shelf lips, chains, wires, or other methods.

Computers and communications equipment-which can overturn if not properly anchored, particularly if they are tall and narrow-are attached to the floor, desks and in some instances to the walls to resist overturning forces.

LS Raised floors with access to computer wiring, are braced to resist lateral forces. In some areas, the bracing is not fully adequate and these raised floors could fall to the structural slab. In some corridors, we found floor tiles that are loose due to the continuous pedestrian traffic and need to be reattached.

5.3 Seismic Analysis of the Building Structure

We direct our efforts here to evaluating the building's general, preliminary seismic risk according to the scope of work. The expected results are meant to be used in assessing the importance of building deficiencies on a conceptual basis. They are the basis for any future seismic rehabilitation plan.

Our analysis follows the methods of the NEHRP Handbook for the Seismic Evaluation of Existing Buildings, FEMA 178/June 1992, as modified by Air Force Engineering Technical Letter (ETL) Structural Evaluation of Existing Buildings for Seismic and Wind Loads.

For this purpose, we modeled the building for a computer-based static finite element analysis. The applied loading was an equivalent pseudo-static load. We used the ETABS computer program (Reference 6). The results of the analysis are discussed in this section. The analysis computer output is presented in Appendix C.

The analysis objective is to deal with the evaluation statements in the field work questionnaires, that are found to be false, and therefore require additional analysis. The analysis procedure consisted of the following steps:

1. Calculate the building weights.

2. Calculate the building period.

3. Calculate the lateral force on the building.

4. Distribute the lateral force over the height of the building and calculate the story shears and overturning moments.

5. Distribute the story shears to the vertical resisting elements in proportion to their relative stiffnesses.

reaction diagrams for diaphragms and for the vertical resisting elements (forces for diaphragm, wall and frame analysis are taken from these diagrams).

b. Calculate shearing stresses and chord forces in the diaphragm,

c. Analyze the vertical components (walls and frames) and find the story deflections and the member forces and deflections.

d. Calculate total forces or deflections according to the specified load combinations.

Since original design calculations were not too clear, we waived the possibility of using a scaling factor to relate the original design base shear to the base shear of this calculation. Our option was to perform an analysis of the entire structure under the prescribed lateral loads. This included checking the adequacy of the load paths, the lateral-force-resisting components, and the details.

Our analysis also included the determination of force level, horizontal distribution of lateral forces, accidental torsion, drift, and overturning. In summary, the analysis of the building covered the following:

- Base Shear: The seismic base shear as it becomes the basic seismic demand on the building.

- Period: The approximate fundamental period of the building.

- Direction of Seismic Forces: Assumption that seismic forces will come from any horizontal direction.

- Uplift: The effects of uplift at the foundation soil level.

- Combination of Structural Systems: The effects due to the combination of structural systems.

- Vertical Distribution of Forces: The vertical distribution among structural members of the horizontal seismic forces induced at any level.

- Horizontal Distribution of Shear: Distribution of the story shear to the various vertical elements of the lateral-force-resisting system in proportion to their rigidities.

- Horizontal Torsional Moments: The increased shears resulting from horizontal torsion. The minimum assumed displacement of the center of mass was estimated to equal 5 percent of the dimension at that level measured perpendicular to the direction of the applied force.

- Overturning: The overturning effects caused by earthquake forces.

- Foundations: The foundation capability of transmitting the base shear and the overturning forces from the structure into the supporting soil. The short-term dynamic nature of the loads was taken into account in establishing the soil properties.

- Soil Capacities: The capacity of the foundation soil in bearing and the capacity of the soil/foundation interface to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For the load combination, including earthquake, the soil capacities to resist loads at acceptable strains considering both the short

- Structural Materials: The strength of concrete foundation components subjected to seismic forces alone or in combination with other prescribed loads.

The seismic performance of an existing building is influenced by many factors including the seismicity of the area in which the structure is located, the materials of construction, the height and geometric form, the structural framing system employed and whether or not a viable lateral force resisting system exists. FEMA 178 recommends a systematic evaluation of all of these factors such as exterior wall construction, roof diaphragm, as well as other factors relating to non-structural items such as ceilings, partitions, mechanical electrical equipment and parapets.

Evaluation statements pertaining to building Type 4: steel braced frame, including necessary calculations were completed and are included in Appendix C.

Equivalent static force design procedure of NEHRP Section 2.4.3.1 (FEMA 178) was used in determining the total base shear. Total lateral seismic force generated by a building above its base is computed according to the formula.

$$V = Cs \times W$$

where:

Cs	=	the seismic design coefficient = 0.67 [1.2 Av S / R T2/3]
<	=	0.85 [2.5 Aa/R] = 0.17 (controls)
W	=	the total seismic dead weight
Aa	=	effective acceleration coefficient in Figure 2.1a which equals 0.4 for the site.
Av	=	the peak velocity-related acceleration coefficient given in Figure 2.1c which equals 0.4 for the site
S	=	the site coefficient given in Table 2.1 (2.0 assumed)
R	=	a response modification coefficient from Table 2.4.3.1
	=	5 for concentrically braced steel frames
T	=	the fundamental period of the building estimated as 0.4 sec as provided in the calculations
hn	=	the height in feet above the base to the highest level of the building, 100 ft
L	=	the overall length of the building in feet, 258 feet

Calculations for masses were prepared and load distribution to different elements of the structure was performed using the computer program ETABS (Reference 6) and hand calculations.

In addition to the lateral force resisting system, the conceptual analysis addresses other building elements such as nonstructural architectural and mechanical elements (e.g., appendages, exterior cladding, and equipment).

On a qualitative basis, we identified some specific deficiencies without any calculation. These are general concerns (e.g., an adjacent building that is too close) or element concerns (e.g., a lack of bracing or a connection).

As identified in the building evaluation questionnaire, parts and portions of structures, permanent nonstructural components, and equipment supported by a structure and their attachments were also conceptually evaluated to verify their capacity for resisting seismic forces. Because the structural failure of nonrigid equipment could cause a life-safety hazard, we also conceptually evaluated these sorts of equipment.

We recommend that as part of a future building seismic rehabilitation program at OAS, further analysis of these elements be performed to include nonstructural architectural and mechanical elements and equipment. All attachments or appendages, including anchorages and required bracing, should be further evaluated for their reaction to seismic forces.

5.5 Final Evaluation

Upon completion of the field work and the analysis, we reviewed the evaluation statements in FEMA 178 guidelines and the responses to these statements to ensure that all of the concerns had been addressed.

We assembled and reviewed the results. The analysis, some calculations and the simplified finite element model are presented in Appendix C. Some results of the frequency analysis and the pseudostatic analysis are highlighted below.

Critical member stress ratios are as follows: Q/C = Applied Force / Capacity

	Q/C
• Axial stress in diagonal braces (first floor)	1.98
• Slabs	1.98
• Foundation piles in compression	1.26
• Foundation piles in tension	3.77

In addition, our analysis shows that:

- The first fundamental period of lateral vibration is: T lat = 0.4 sec.

Based on a review of the complex mix of qualitative and quantitative results of the analysis and the observed deficiencies, our final evaluation of the building leads us to believe that it has a propensity to partial seismic failure of both structural and nonstructural components.

It is our opinion that structure of Building 1003 does not meet current code requirements for earthquake resistance. The data from our analysis confirms the recommendations given in previous studies about the need to improve the lateral resistance of the building steel frame. As it was demonstrated at the time of the 1989 Loma Prieta earthquake, significant damage to the structure should be expected in the event of a strong earthquake.

From the original construction data (Reference 10), the structure appears to have been designed in 1967 using seismic capacity requirements consistent with the state of industry at that time which was regulated by various codes, including the 1964 Uniform Building Code. The equivalent seismic base shear coefficient used for the design appears to have been on the order of 0.1 g. Current codes such as the 1994 UBC and others (References 9, 11, 12 and 13) as well as the FEMA 178 guidelines being used at this time result in seismic base shear coefficient about 80% higher.

The building 1003 structure, as is typical of structures of that vintage does not meet current requirements and lacks adequate strength to resist realistic strong earthquakes. It then comes as no surprise that the 1989 Loma Prieta earthquake caused significant damage to the structure as described in a previous partial structural inspection study performed by EG&G Idaho (Reference 15).

The report states that considerable damage to the structure took place in the 1989 Loma Prieta Earthquake. It was estimated that approximately 90% of the east/west lateral load resisting system at the first and second floor levels was severely damaged. The report also identified a total of 32 connections as being severely damaged.

There is indication (Reference 15) that 11 connections were being repaired in 1992. However we have not been able to find evidence of any strengthening program for the structure.

Reference 19 provides indication that 22 more joints were repaired following the initial effort for the 11 connections. It is assumed that the original design capacity of the structure has been restored to the levels of 1967 but not to current levels.

Reference 14 provides a structural upgrade project description with various seismic retrofitting alternatives. Among these alternatives are:

- Base isolation seismic upgrade
- Energy dissipation with shear panels
- Energy dissipation with braces
- Exterior steel panels
- Addition of exterior moment frame
- Addition of exterior braced frame
- Addition of interior bracing
- Strengthening of Roof Diaphragm
- Strengthening of Floor Diaphragms

the other is for Operation protection (OP). The total costs (LSO + OP) given in 1993 for the various alternatives ranged from $12 million to $ 86 million. The structural portion of the costs was between $ 5 million to $ 11 million. To the best of our knowledge, there has been no upgrade program implemented to date.

From the geological standpoint, it is important to avoid resonance conditions between the building and the ground when making plans for a future seismic retrofitting of the building. More specifically, it is crucial that the fundamental period of ground shaking does not essentially match the natural period of lateral vibration of the building structure.

For this purpose, a verification of the HLA's ground period is recommended. The shape of the HLA's ground response spectra shows the natural ground period to be within 0.2 and 0.3 seconds. This verification can be accomplished either experimentally in the field by performing a geophysical survey, and/or analytically by analyzing the ground as a multistory shear structure.

Also, it should be noted that the building is sitting at a location with potential for seismically induced liquefaction. This is a localized potential for the northwest part of the building. In addition, the building is stressed and excessively loaded, especially at the roof/floor diaphragm levels, perimeter frames and foundation. The combination of all these conditions is not seismically appropriate.

We understand that the building is currently categorized as an essential facility which requires it to remain undamaged and operational after earthquakes. But based on our findings, it does not satisfy the requirements imposed by this building occupancy category, because it cannot sustain a strong earthquake without experiencing damage.

5.6 List of Deficiencies

We have addressed the overall conceptual seismic status of the evaluated building with respect to structure, foundation, site geology, and nonstructural elements. As described throughout the report, the results of our evaluation show whether or not the building elements meet established seismic-resistance requirements.

For those elements not meeting the specified acceptance criteria, our evaluation assesses the relative hazard or seriousness of the deficiencies. We have listed all such deficiencies that were identified. These deficiencies are the shortcomings of the building that must be remedied in order to change evaluation statement responses from false to true.

The deficiencies are classified as structural and nonstructural. Structural deficiencies are directly related to the building structure capacity to support seismic forces. Nonstructural deficiencies are related to the nonstructural building components or parts of other equipment or structures in the building that do not provide the building with any capacity to support seismic forces-such as light fixtures, ceilings, partition walls, roof-mounted equipment, roof catwalks and other miscellaneous items.

We have also ranked the deficiencies according to degrees of importance in the seismic load path and building stability and according to the hazard level that they represent. The complete list of deficiencies is shown in Appendix C.

In the previous section, we developed a list of deficiencies that were identified for Building 1003. These deficiencies must be remedied in order for the building to become seismically adequate. This list is presented in Appendix C.

Based on possible approaches to seismic rehabilitation, we offer a preliminary recommendation of the cost for mitigation work of these building deficiencies. The estimated cost is to be used for the OAS program level budget and decision-making.

The estimated costs are based on guidelines given in Typical Costs for the Seismic Rehabilitation of Existing Buildings, Volumes I and II, FEMA-156/July 1988 and FEMA-157/September 1988 (Reference 7), and our experience with performing seismic retrofitting work under today's conditions. These costs are not intended to be final cost estimates for rehabilitation work. Rather, our intention is to give OAS a cost for budgetary purposes to weight the economics of different options available for the building.

The OAS will have to face the options of implementing various levels of rehabilitation, downgrading of the building's occupancy category, or simply doing nothing to the building. Abandonment and demolition of the building is another option that seems very unlikely to happen.

Any estimated cost has two components: direct and indirect costs. The direct costs have been calculated based on costs for the building's structural type and cost indexes available. Indirect costs such as relocation of occupants or business interruptions are not included and could be substantial due to the fact that the repair work will be extensive and will require vacating offices for an undetermined period of construction. The indirect cost is not provided since the OAS will incorporate this cost at the time of the overall analysis and inventory of OAS buildings.

Following is a summary of the estimated cost for Building 1003. A more detailed description is shown in Appendix C.

6.1 Building 1003 Seismic Retrofitting Costs

Deficiency Mitigation	Cost
Structural Costs	11,477,000
Non-Structural Costs	263,000
Finishing Costs	587,000
Project Costs (A/E, CM, etc.)	2,466,000
TOTAL	14,793,000

Our conclusions and recommendations in regards to Building 1003 are based on our review of available OAS records, our field visits and inspections, and the analysis presented in the preceding sections.

We believe that this building is at risk. It has a propensity to partial seismic failure of structural and nonstructural components. In addition, since this building is located nearby Building 1001, the concern for the potential for seismically induced liquefaction also exists (Reference 8). We recommend to address this concern for both buildings simultaneously.

We recommend that prior to the start of a comprehensive Building Seismic Rehabilitation Program at OAS, an updated geotechnical study based on new data should be conducted to include the monitoring of periodic fluctuations in the groundwater table, additional sampling of liquefaction prone strata, and the laboratory cyclic triaxial testing on representative samples for undrained/drained shear strength of the questionable strata under simulated load conditions induced by the maximum credible earthquake.

If the study confirms that the liquefaction potential still exists, possible avenues of mitigation may include, among other methods, additional and deeper building foundations or soil densification by grout injection. This work should be devised such that any interruptions to the continued use of the facility during the grouting operation is minimized.

The building appeared to be well maintained and in good condition. The structure is well constructed but contains deficiencies which could cause it to have extensive structural damage or collapse in a major seismic event. More specifically, our analysis and evaluation of the building have confirmed that the structure lacks an adequate lateral-force-resisting system. The diagonal braces and foundations are overstressed. There are no in-plane braces in the floor and roof slabs.

Following the 1989 Loma Prieta earthquake, some reports documented the damage suffered by the building, specially at the frame connections. Dislocation and failure of certain connections were observed by EG&G Idaho (Reference 15). Cracks were observed in a gusset plate at the connection of diagonal brace to a column. Repairs were made in 1992 to restore the original design capacity. Various studies recommended the strengthening of the structure. However, there is no evidence that a strengthening construction program was ever performed.

The steel frame connections problems that were found are in line with similar damage of steel frame buildings located at California State University in Northridge resulting from connection failures during the Northridge Earthquake of 1994 and Kobe Earthquake. As a consequence, the American Institute of Steel Construction (AISC) has modified its recommendations for welded connections to withstand seismic loads.

The building cannot sustain a strong earthquake without experiencing damages. It does not satisfy requirements imposed by the current building occupancy category. The building is an essential facility which requires to remain undamaged and operational after earthquakes.

This building will have to undergo seismic rehabilitation if the current building occupancy is not changed. An alternative to seismic rehabilitation is to downgrade the occupancy category to a building that will experience damage, unlikely to cause collapse, but will not be operational and will need repairs after an

The structural performance of this structure can be significantly improved by improving the lateral load resisting system as follows:

- Strengthen the diagonal bracing system.
- Provide in-plane diagonal brace for the roof and floor slabs.
- Provide a stronger foundation.

In regards to the roof level, additional in-plane diagonal bracing should be provided in the building frame underlying the specific location where excessive roof loading has been imposed due to the installation of heavy mechanical equipment, since the building was constructed.

In regards to strengthening the lateral-force capacity of the building, the need of diagonal bracing for the building steel frame or additional shear walls at the ground level of the structure is recommended. A further analysis may also indicate the need to" tie" this building to other adjacent structures together.

It will be useful to recommend nonlinear earthquake response procedures to evaluate the seismic safety of this building under the influence of a maximum credible earthquake. All the prior work done by others (References 14 through 18) so far in assessing seismic safety of this building is based on linear elastic seismic response analysis, which has limited application for analyzing the effects of the maximum credible earthquake. Also, given the access limitations and difficulties in inspecting the connections of this building, there are certain NDE procedures available using radar and X-ray techniques to examine whether the affected joints are still serviceable.

Other work to retrofit nonstructural building elements is also recommended. The building deficiencies and the rehabilitation work needed to bring the building to seismic safety have been identified. The list is shown in Appendix C. The cost for rehabilitation is estimated to be in the order of $ 14,793,000.

In closing, we want to stress the fact that the costs of the rehabilitation work for this building have been calculated for budgetary purposes only. These costs include structural, nonstructural finishing and project costs. Indirect costs are not included. The OAS will incorporate the indirect costs at the time of the overall analysis and inventory of OAS buildings.

We have achieved the objective of the OAS, which is to evaluate Building 1003 and comply with Presidential Executive Order 12941, Seismic Safety of Existing Federally Owned or Leased Buildings. The report is accompanied with a tabulated data chart which will be added to the general building inventory data base of OAS.

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2. NEHRP Handbook for the Seismic Evaluation of Existing Buildings, FEMA 178/June 1992.

3. Structural Evaluation of Existing Buildings for Seismic and Wind Loads, Engineering Technical Letter (ETL), Air Force Civil Engineering Support Agency, September 1994.

4. Studies for the Seismic Reinforcing of Building 1003:
   a) Final Report for Seismic Design Criteria for Building 1003, Harding Lawson Associates, 1993.
   b) Geotechnical Investigation UPS Building, Onizuka Air Force Base, Sunnyvale, CA , Dames and Moore, October 6, 1993
   c) Summary of Pier Load Test Results, Emergency Utility Building, Onizuka Air Force Base, Sunnyvale, CA, Kaldveer Associates, September 16, 1991.

5. Ground Motions and Soil Liquefaction During Earthquakes, Seed & Idriss, 1982, Earthquake Engineering Research Institute.

6. ETABS, Version 6, Computers and Structures Inc., Berkeley, California, 1995.

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8. Seismic Evaluation for Buildings 1001, 1002, 1010 and 1013 at Onizuka Air Station, Mariscal Engineering, March 1997.

9. Uniform Building Code (UBC), 1994 edition.

10. Building 1003 Drawings and Structural Calculations dated 1967.

11. Steel Construction Manual, Ninth Edition, American Institute of Steel Construction.

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13. Recommended Lateral Force Requirements and Commentary, Structural Engineers Association of California, San Francisco, CA ,1996.

14. Onizuka Air Force Base, Building 1003, Seismic Study, Cygna Group, March 31, 1993, including drawings.

15. Onizuka Air Force Base, Building 1003, Structural Inspection, EG&G Idaho, Inc., February 14, 1992.

16. Seismic Evaluation, Building 1003, Onizuka Air Force Station, Holmes & Narver, Inc., June 1987.

17. Earthquake damage, Building 1003 Onizuka Air Force Base, Sunnyvale, CA., Preece/Goudie & Associates, October 17,1991.

18. Structural Inspection Onizuka A.F.B., Building 1003, SAI Engineers, October 24,1991.

19. Building 1003 Undergoing Repairs, Onizuka Orbiter Newsletter, February 10, 1992.

APPENDIX A

SCOPE OF WORK

STATEMENT OF WORK
FOR

SEISMIC EVALUATION OF BUILDING 1003, OAS

Project WMSJ 96-1054

ONIZUKA AIR STATION
SUNNYVALE, CALIFORNIA

AUGUST 1997

Prepared by:
750MSS/CEE

United States Air Force
ONIZUKA AIR STATION
SUNNYVALE, CALIFORNIA

THIS ATTACHMENT, INCLUDING COVER SHEET CONTAINS for

ARCHITECT-ENGINEER SERVICES

Project No. WMSJ 96-1054

SEISMIC EVALUATION OF BUILDING 1003

at

ONIZUKA AIR STATION

13 August 1997

1.0 OBJECTIVE

The objective of this study is to initially evaluate building #1003 at Onizuka Air Station for potential earthquake-related risk to human life posed by a building or building component. This portion of the study is to be used in the overall analysis and inventory of OAS buildings to comply with Executive Order 12941, Seismic Safety for Existing Federally Owned or Leased Buildings. The form and content of the evaluation shall follow the method of the NEHRP Handbook for e Seismic Evaluation of Existing Buildings (FEMA-178/June 1992) as modified by Air Force Engineering Technical Letter (ETL) Structural Evaluation of Existing Buildings for Seismic and Wind Loads. Geologic hazards, the building's structural system, foundation, and non-structural elements shall be evaluated. The results of the initial evaluation shall be presented in a report which lists those elements that do not meet the basic acceptance criteria, compares the demand to the capacity of those elements, assesses the consequences of the failure of the elements with high demand to capacity ratios, and states the Architect-Engineer (A-E's) judgment of the potential life safety hazard. In addition, this study will evaluate geologic hazards of building #1003 and provide a "program level" estimate of the cost for the recommendations to rehabilitate or upgrade building.

2.0 SCOPE

Work scope and the evaluation criteria are contained in the NEHRP Handbook for the Seismic Evaluation of Existing Buildings (hereafter referred to as FEMA-178) except as modified by ETL Structural Evaluation of Existing Buildings for Seismic and Wind Loads. The scope of the total A & E efforts shall consist of, but not be limited to, a complete site survey and investigation of building #1003. The A-E shall provide all services , tools, equipment, and transportation required to evaluate building #1003 and to prepare the written report. The final report shall address overall conceptual seismic status of building #1003 with respect to structure, foundation, site geology, and non-structural elements. The final report

3.0 REFERENCE DOCUMENTS

A) The following documents and publications referenced in these documents form a part of this statement of work:

3.1 Inventory, Screening, Prioritization, and Evaluation of Existing Buildings for Seismic Risk, Engineering Technical Letter, Air Force Civil Engineering Support Agency, 18 August 1993.

3.2 Structural Evaluation of Existing Buildings for Seismic and Wind Loads, Engineering Technical Letter, Air Force Civil Engineering Support Agency, September 1994 (Draft).

3.3 Seismic Design for Buildings, AFM 88-3, Chap 13, October 1992.

3.4 ICSSC RP 5/October 1995, ICSSC Guidance on Implementing Executive Order 12941 on Seismic Safety of Existing Federally Owned of Leased Buildings, U.S. Department of Commerce, Building and Fire Research Laboratory, National Institute of Standards and Technology.

B) The following documents are to be used as reference only (not to be compiled with specifically):

3.5 FEMA 156/July 1988, Typical Costs for the Seismic Rehabilitation of Existing Buildings, Volume I - Summary. Federal Emergency Management Agency.

3.6 FEMA 157/September 1988, Typical Costs for the Seismic Rehabilitation of Existing Buildings, Volume II - Supporting Documentation. Federal Emergency Management Agency.

3.7 FEMA 178/June 1992, NEHRP Handbook for the Seismic Evaluation of Existing Buildings, Federal Emergency Management Agency.

3.8 FEMA 222/January 1992, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Part 1: Provisions, Federal Emergency Management Agency. (Includes Maps).

3.9 FEMA 223/January 1992, NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, Part 2: Commentary, Federal Emergency Management Agency.

4.0 BACKGROUND

Onizuka Air Station is a satellite testing and control facility located at Mathilda Avenue and Moffett Park Drive in Sunnyvale, California adjacent to the Lockheed Martin Company. The buildings to be evaluated is located on the 22 acre campus of Onizuka Air Station. Building #1003 is the "Risk Group A" building on Onizuka Air Station. This building was selected for evaluation based on screening criteria in ETL 93-3

5.1 Geotechnical Evaluation

Evaluate the Onizuka Air Station site for the geologic hazard listed in FEMA-178, Section 9.3. This is a requirement of ETL 93-3 (paragraph 5.7.1). The intent of this portion of the evaluation is to identify geologic hazards that might affect the building included on this study, on the basis of the geotechnical characteristics as shown on existing documents. The evaluation shall be a minimal assessment of the property to complete the ICSSC database using existing site data. No soil borings or other underground soil samplings are needed nor requested for the purpose of this study.

5.2 Structural Evaluation

For building #1003, address the general set of evaluation statements or the appropriate statement(s) for the common building type(s) applicable to the building. This evaluation shall be of a conceptual nature. The evaluation statements for geotechnical hazards/foundations and non-structural elements shall be addressed for the building. Modifications to work scope or criteria in references are identified by section. If the statement "No additional requirements" appears below the task heading, follow the references' guidance as written. Other information such as available data and constraints affecting the work are included in the following sections.

5.3 Site Visit and Data Collection

5.3.1 Access to the building at Onizuka Air Station is "restricted." See Section 8.3 of this Statement of Work for requirements for access to "restricted" areas.

5.3.1.1 Photographing is prohibited on Onizuka Air Station without permission from station Security Police. Notify the Government's engineer two weeks in advance to arrange for permission to take photographs or to have Bertha Roman take photographs.

5.3.1.2 Geotechnical reports for Onizuka Air Station are available for review in the Engineering office. A site specific response spectra has also been developed for the Onizuka Air Station.

5.3.1.3 The record drawings for building #1003 at Onizuka Air Station to be evaluated are in the Engineering office. The current configuration of this building is depicted in various sets of drawings that are complete. There is no need to produce as-built drawings for this study.

5.3.2 Selection and Review of Evaluation Statements

No additional requirements.

5.3.3 Follow-up Field Work

Fieldwork is limited to that work necessary to address the evaluation statements which can be accomplished using non-destructive testing methods.

Assess the relative importance of deficiencies on a conceptual basis. This assessment will be upgraded in the future to a full evaluation as required by FEMA-178 Section 2.4.12, which is not in this scope of work at this time.

5.3.5 Final Evaluation

No additional requirements.

5.3.6 The Final Report

Prepare a final evaluation report with sections for the building and sections for their geologic hazard evaluation at Onizuka Air Station. The building sections shall include conceptual and "qualitative answers". The final report shall have basic conceptual recommendations for priority for mitigating deficiencies. The recommended priority shall cover the building evaluated. Include recommendations for mitigation and possible approaches to rehabilitate the building and the program level cost estimate in the report. The final report will not be in full compliance with FEMA 178/June 1992 since this will be the scope of final work. Include in the final report a table of information which matches the table format in the ICSSC Guidance on Implementing EO 12941. This table must be prepared and presented using Microsoft Excel, version 5.0a (see ICSSC RP5).

6.0 DELIVERABLES

Submit three copies of the report for government review. A "review" and a "revised" version of the report shall be submitted. The "revised" version shall include the corrections, clarifications, and additions resulting from the Government review of the report.

7.0 REVIEWS AND MEETINGS

7.1 Reviews

The A-E shall submit the report for Government review. The Government will return written comments to the A-E ten working days after receiving the deliverables. The A-E shall provide written responses to comments and revise the report if necessary. Responses to comments and the revised report shall be submitted within 15 days after receiving the Government's comments. The Government will perform a back-check review to insure all comments have been addressed or incorporated in the final report. If a lack of compliance is noted, an additional back-check review and deliverable will be required without additional cost to the Government.

7.2 Meetings

Government to ascertain the progress of the project.

8.0 SPECIAL CONSIDERATIONS

8.1 Contract Document Verification and Quality Control

The A-E shall establish a system of in-house peer reviews for quality control of the evaluations, including calculations, to ensure compliance with the task's requirements.

8.2 Schedules and Reviews

8.2.1 Schedules

The work shall be completed with 90 calendar days. Within 15 working days after the execution of delivery orders, the A-E shall submit a schedule for all submittals, Government reviews, and A-E revisions. The Government will provide the submittal delivery date with each delivery order. A-E will not modify the schedule after approval unless the revision is approved by the Government.

Progress schedules shall be submitted every two weeks clearly indicating tasks completed and the percent of the work complete. This information shall also be provided to the Government's Engineer in a telephone conference.

8.2.2 Submittal Reviews

The A-E shall allow ten working days from the date of Government receipt for the Government to review the submittal.

8.3 Access and Security

Onizuka Air Station is engaged in the operational testing and control of Department of Defense satellites. Access to many areas of Onizuka Air Force Base is restricted. Advance notice to the Base Civil Engineer is required to arrange access and security escorts. This may require furnishing some personal data on the personnel requiring access and may lead to some inconvenience or access delays. Access to environmentally sensitive or contaminated areas will be arranged by the Base Civil Engineer after areas are cleared from potential health risks to A/E staff.

8.4 Engineering Calculations

Calculations required for the evaluation shall be prepared and stamped by a professional engineer licensed in the discipline related to the work.

8.5 Administrative Service

title, current project phase estimated and actual completion dates, project estimated and actual completion dates and remarks regarding progress.

*** END OF SOW ***

APPENDIX B

REVIEWED DOCUMENTS OF RECORD

EXISTING STUDIES AND REPORTS

1. Seismic Evaluation for Buildings 1001, 1002, 1010 and 1013 at Onizuka Air Station, Mariscal Engineering, March 1997.
2. Final Report for Seismic Design Criteria for Building 1003, Harding Lawson Associates, 1993.
3. Geotechnical Investigation UPS Building, Onizuka Air Force Base, Sunnyvale, CA , Dames and Moore, October 6, 1993.
4. Summary of Pier Load Test Results, Emergency Utility Building, Onizuka Air Force Base, Sunnyvale, CA, Kaldveer Associates, September 16, 1991.
5. Onizuka Air Force Base, Building 1003, Seismic Study, Cygna Group, March 31, 1993, including drawings.
6. Onizuka Air Force Base, Building 1003, Structural Inspection, EG&G Idaho, Inc., February 14, 1992.
7. Seismic Evaluation, Building 1003, Onizuka Air Force Station, Holmes & Narver, Inc., June 1987.
8. Earthquake damage, Building 1003 Onizuka Air Force Base, Sunnyvale, CA., Preece/Goudie & Associates, October 17,1991.
9. Structural Inspection Onizuka A.F.B., Building 1003, SAI Engineers, October 24,1991.
10. Structural Calculations for Building 1003 Addition I, Mission Control Complex, GKT Consulting Engineers, Inc., August 17, 1981.
11. Building 1003 Drawings and Structural Calculations dated 1967.

Oakland, CA 94612
(510) 839-4330

ONIZUKA AIR STATION
LIST OF EXISTING SOILS STUDIES

ITEM #	DESCRIPTION	BY	DATE
1	Final Report for Seismic Design Criteria for Bldg 1003	Harding Lawson Associates	1993
2	Pier Load Test Report	Kaldveer Associates	1991
3	Geotechnical Investigation for UPS Bldg at OAS	Dames & Moore	1993
4	Geotechnical Investigation for Emergency Utilities Bldg	Brown & Root	1989
5	Soil Investigation for Bldg 1005 Addition & New Mission Control Bldg	Harding Lawson Associates	1982
6	Soil Investigation for Mission Control Complex	Harding Lawson Associates	1981
7	Soil and Foundation Investigation for Satellite Test Center Expansion	Harding Associates	1966
8	Development Control Center, Soil Boring Location Plan and Profiles	Ralph M. Parson	1959

D R A W I N G   I N D E X

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Drawing Index
Site and Vicinity Map
Plot Plan
Demolition, Grading, and Drainage Plan
Overall Utility Plan
Lawn Sprinkler Plan and Details
Miscellaneous Details
Soil Borings - Plan and Logs
Landscape Plan and Details
Fencing Details
ARCHITECTURAL DRAWINGS
Architectural Abbreviations, Symbols & Legends
First Floor Plan
Wall Construction & Finish Schedule - First Floor
Door Schedule - First Floor
Ceiling Plan - First Floor
Second Floor Plan
Mezzanine Plan
Wall Construction & Finish Schedule - Second Floor, Sheet No. 1
Wall Construction & Finish Schedule - Second Floor, Sheet No. 2
Door Schedule - Second Floor
Ceiling Plan - Second Floor
Third Floor Plan
Wall Construction & Finish Schedule - Third Floor - Sheet No. 1
Wall Construction & Finish Schedule - Third Floor - Sheet No. 2
Wall Construction & Finish Schedule - Third Floor - Sheet No. 3
Door Schedule - Third Floor - Sheet No. 1
Door Schedule - Third Floor - Sheet No. 2
Ceiling Plan - Third Floor
Fourth Floor Plan
Wall Construction & Finish Schedule - Fourth Floor - Sheet No. 1
Wall Construction & Finish Schedule - Fourth Floor - Sheet No. 2
Wall Construction & Finish Schedule - Fourth Floor - Sheet No. 3
Door Schedule - Fourth Floor - Sheet No. 1
Door Schedule - Fourth Floor - Sheet No. 2
Ceiling Plan - Fourth Floor
Roof Plan
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South Elevation
East Elevation
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Building Section "B-B"
Stairs No. 1 & 2 Plans & Sections
Stairs No. 3 & 4 Plans & Sections
Elevator Plans & Details
Air Conditioning Chase & Plenum - Section "C-C"
Main Cable & Signal Cable Chase - Plans & Sections
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Interior Elevations
Interior Elevations
Raised Floor Details
Door & Window Details

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B 91092	1117092	53	A-43	Door Details
B 91093	1117093	54	A-44	Interior Details
B 91094	1117094	55	A-45	Interior Details
B 91095	1117095	56	A-46	Interior Details
B 91096	1117096	57	A-47	Exterior Details
B 91097	1117097	58	A-48	Roof Details
B 91098	1117098	59	A-49	Floor Plans - Bldg. #1 Modifications & Bldg. Connections #1 & #2
B 91099	1117099	60	A-50	Ceiling & Roof Plans - Bldg #1 Modifications & Bldg. Connections #1 & #2
B 91100	1117100	61	A-51	Elevations & Sections - Bldg #1 Modifications & Bldg. Connections #1 & #2
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B 91103	1117103	64	S-2	Pile Schedule & Details
B 91104	1117104	65	S-3	Foundation Plan
B 91105	1117105	66	S-4	Pile Cap Details
B 91106	1117106	67	S-5	First Floor Slab Plan
B 91107	1117107	68	S-6	Pedestal & Anchor Bolt Schedule & Details
B 91108	1117108	69	S-7	Foundation Details Sh. No. 1
B 91109	1117109	70	S-8	Foundation Details Sh. No. 2
B 91110	1117110	71	S-9	Second Floor Framing Plan
B 91111	1117111	72	S-10	Mezzanine Framing Plan
B 91112	1117112	73	S-11	Third Floor Framing Plan
B 91113	1117113	74	S-12	Fourth Floor Framing Plan
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B 91121	1117121	82	S-20	Line 4 Framing Elevation
B 91122	1117122	83	S-21	Line 5 Framing Elevation
B 91123	1117123	84	S-22	Line 6 Framing Elevation
B 91124	1117124	85	S-23	Line 7 Framing Elevation
B 91125	1117125	86	S-24	Line 8 Framing Elevation
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B 91127	1117127	88	S-26	Line D Framing Elevation
B 91128	1117128	89	S-27	Stair No. 1 & 2 Framing Plans
B 91129	1117129	90	S-28	Stair No. 3 & 4 Framing Plans
B 91130	1117130	91	S-29	Stair Framing Details
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B 91155	1117155	116	M-6	Control Diagrams System AC-4 & Miscellaneous
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B 91173	1117173	134	M-24	First Floor - Pneumatic Tube & Fire Protection Plan
B 91174	1117174	135	M-25	Second Floor - Pneumatic Tube & Fire Protection Plan
B 91175	1117175	136	M-26	Third Floor - Pneumatic Tube & Fire Protection Plan
B 91176	1117176	137	M-27	Fourth Floor - Pneumatic Tube & Fire Protection Plan
B 91177	1117177	138	M-28	Pneumatic Tube System Schematic
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B 91179	1117179	140	M-30	Plumbing Plan 1st & 2nd Floor Toilet Rooms
B 91180	1117180	141	M-31	Plumbing Plan 3rd & 4th Floor Toilet Rooms
B 91181	1117181	142	M-32	Plumbing Plan Mezz. Plan & Toilet Room Cold Water & Hot Water Piping
B 91182	1117182	143	M-33	Plumbing Sections & Details
B 91183	1117183	144	M-34	Piping & Controls Plan 1st Floor
B 91184	1117184	145	M-35	Piping & Controls Plan 2nd Floor
B 91185	1117185	146	M-36	Piping & Controls Plan 3rd Floor
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B 91197	1117197	158	E-11	Grounding & Cable Trays, Third Floor Plan
B 91198	1117198	159	E-12	Grounding & Cable Trays, Fourth Floor Plan
B 91199	1117199	160	E-13	Grounding & Cable Trays, Mezzanine Plan
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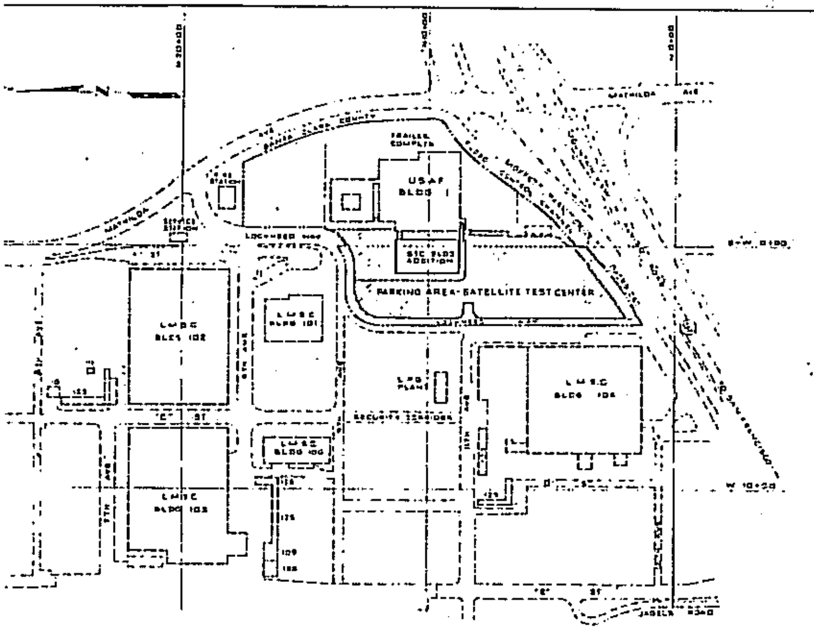
CFD NO	FEC NO	SHT NO	REF NO	
B 91210	1117210	171	E-24	Di
B 91211	1117211	172	E-25	Li
B 91212	1117212	173	E-26	Li
B 91213	1117213	174	E-27	Li
B 91214	1117214	175	E-28	Li
B 91215	1117215	176	E-29	Li
B 91216	1117216	177	E-30	Li
B 91217	1117217	178	E-31	Fl
B 91218	1117218	179	E-32	Ro
B 91219	1117219	180	E-33	Lig
B 91220	1117220	181	E-34	Lig
B 91221	1117221	182	E-35	Lig
B 91222	1117222	183	E-36	Cl
B 91223	1117223	184	E-37	Cir
B 91224	1117224	185	E-38	Tec
B 91225	1117225	186	E-39	Nu
B 91226	1117226	187	E-40	Sc

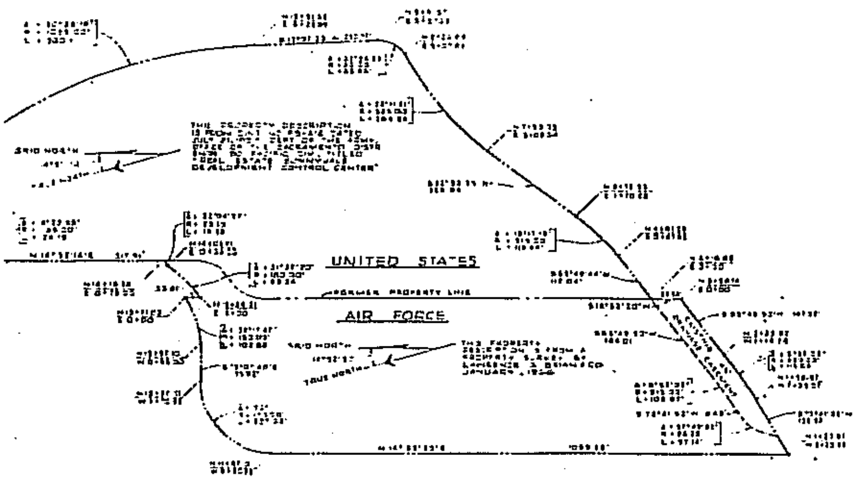
C F BRAUN & CO
ARCHITECT - ENGINEER
ALHAMBRA, CALIFORNIA

R O BALDWIN
STC



SITE PLAN

SCALE: 1"=100'

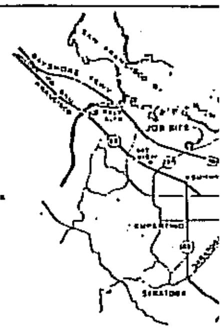


SATELLITE TEST CENTER SITE PROPERTY DESCRIPTION

SCALE: 1"=100'

ABBREVIATIONS

AC	ASPHALT CONCRETE	INV	INVERT
BRKR	BREAKER	JT	JOINT
BLDG	BUILDING	LMSC	LOCKHEED MISSILES & SPACE CO
BM	BENCH MARK	LPS	LIQUID PROPANE GAS
BF	BLIND FLANGE	LS	IRRIGATION SYSTEM
		LBS	POUNDS
CB	CATCH BASIN	MAX	MAXIMUM
CB	CURB	MH	MANHOLE
C-C	CENTER TO CENTER	MIN	MINIMUM
CT	CURB TACK	OC	ON CENTER
CI	CAST IRON	OD	OUTSIDE DIAMETER
CIP	CAST IRON PIPE		
CISP	CAST IRON SOIL PIPE	PCC	PORTLAND CEMENT CONCRETE
CL	CLEAR	PG&E	PACIFIC GAS & ELECTRIC CO
CMP	CORRUGATED METAL PIPE	PI	POINT OF INTERSECTION
CONC	CONCRETE	PIV	POST INDICATOR VALVE
CONN	CONNECTION	PT	POINT
CONST	CONSTRUCT	PSI	POUNDS PER SQUARE INCH
CONT	CONTINUOUS	PVMT	PAVEMENT
DIA	DIAMETER	R or RAD	RADIUS
DEPT	DEPARTMENT	RD	ROOF DRAIN
DW	DOMESTIC WATER	REQD	REQUIRED
DWG	DRAWING	RED	REDUCER
EA	EACH	S	SLOPE
EL OR ELEV	ELEVATION	SD	STORM DRAIN
ELEC	ELECTRIC	SCH	SCHEDULE
ENGR	ENGINEER	SQ	SQUARE
EW	EACH WAY	SPEC	SPECIFICATION
EX OR EXIST	EXISTING	STC	SATELLITE TEST CENTER
EXP	EXPANSION	STA	STATION
		SS	SANITARY SEWER
FDN	FOUNDATION		
FF	FINISH FLOOR	T & B	TOP & BOTTOM
FH	FIRE HYDRANT	TC	TOP OF CURB
FIN	FINISH	TYP	TYPICAL
FL	FLOW LINE		
FT	FOOT OR FEET	OPNG	OPENING
FW	FIRE WATER		
		USAF	UNITED STATES AIR FORCE
GA	GAGE		
GV	GATE VALVE	VCP	VITRIFIED CLAY PIPE
GALV	GALVANIZED	VERT	VERTICAL
HB	HOSE BIB	W/	WITH
HDR	HEADER	WT	WEIGHT
HORIZ	HORIZONTAL	Δ	DELTA = CENTRAL ANGLE
HP	HIGH POINT		
HPFS	HIGH POINT OF FINISH SURFACE	Ø	ROUND



VICINITY

LEGEND

USAF

EXISTING

NEW WORK

DETAIL

SHEET

SECTION

SHEET

SECTION

SHEET OR DETAIL

GRAPHIC SCALES

100 0 200 400 FT

SCALE 1"=200'

100 0 100 200 FT

SCALE 1"=100'

RECORD DRAWING

C F BRAUN & CO

ARCHITECT - ENGINEER

ALHAMBRA, CALIFORNIA

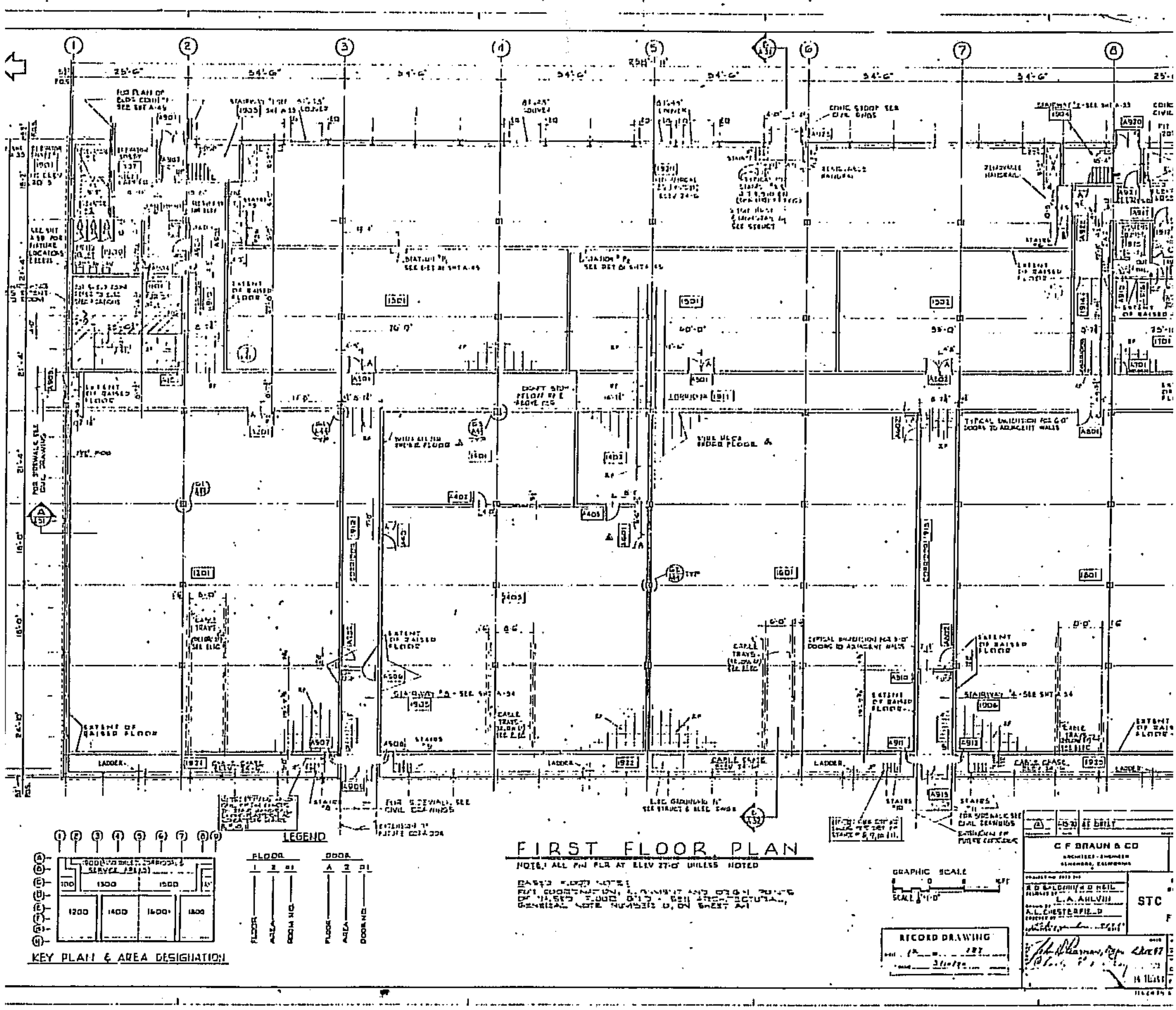
R A BAILE

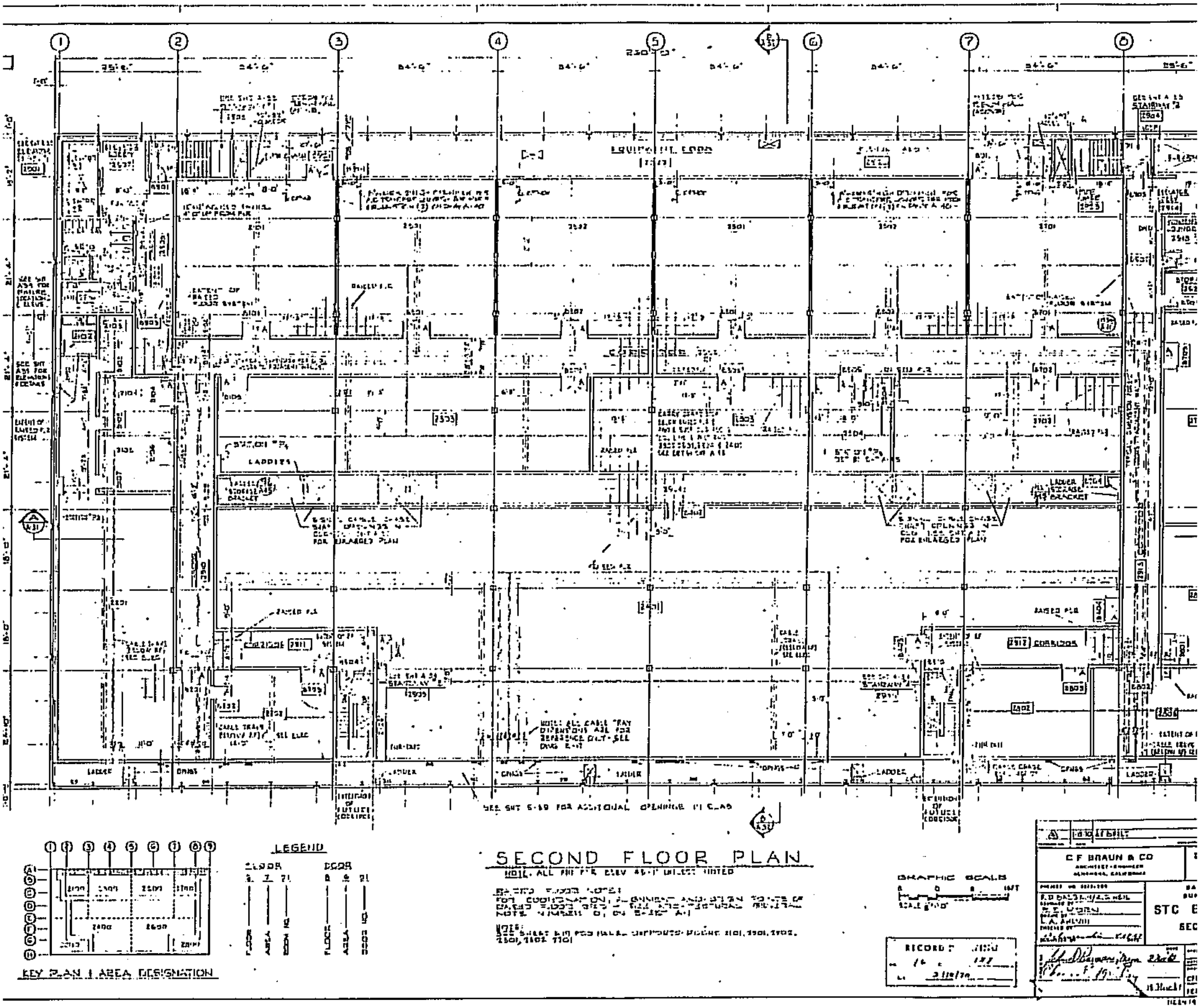
A DUNDA

R.W. GIBBONS

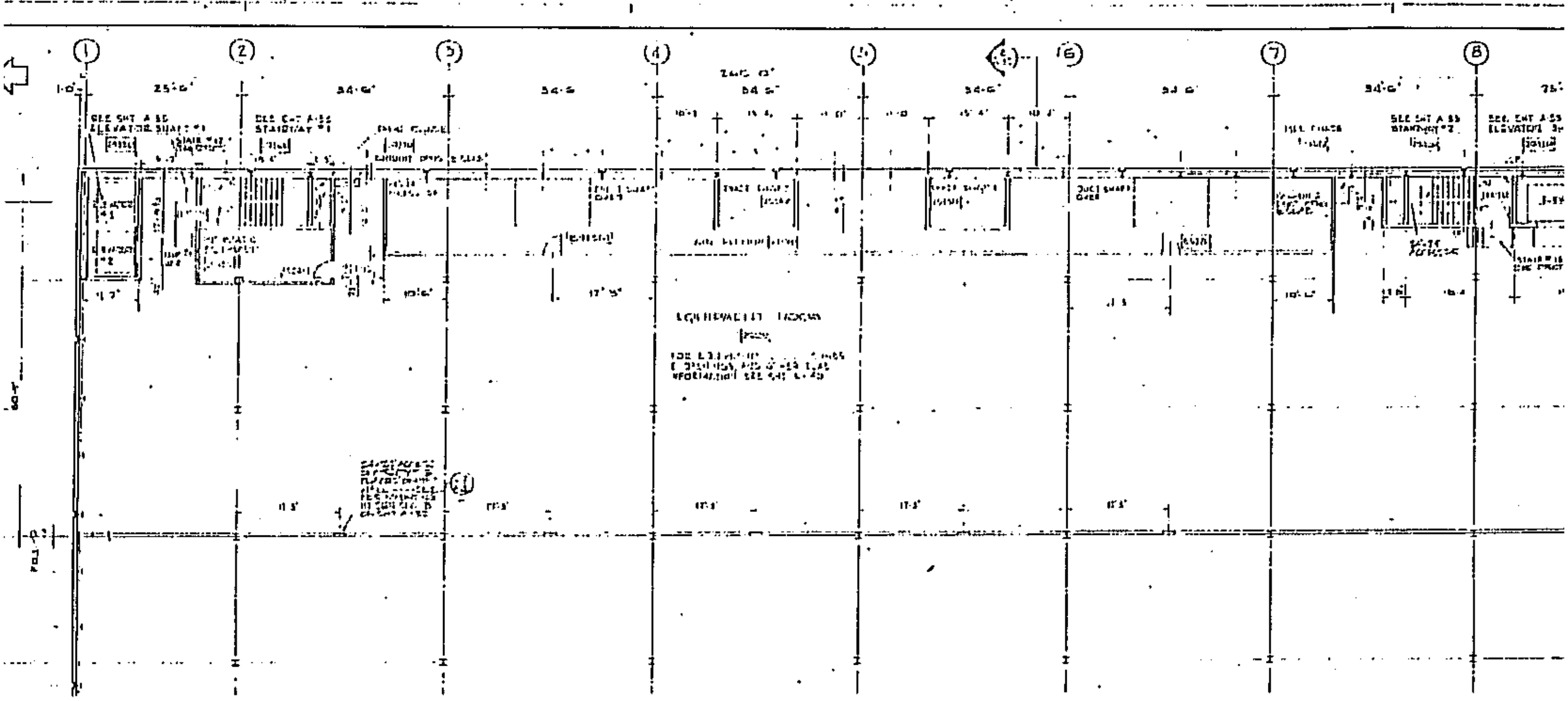
STC B

SITE


FIRST FLOOR PLAN
NOTE: ALL FIN FLR AT ELEV 27'-0" UNLESS NOTED
KEY PLAN & AREA DESIGNATION
LEGEND
FLOOR
AREA
ROOM NO.
DOOR
DOOR NO.
GRAPHIC SCALE
RECORD DRAWING
AS BUILT
C F BRAUN & CO
L.A. AHLVIN
A.L. CHESTERFIELD
STC
LADDER
STAIRS
CABLE CHASE
EXTENT OF RAISED FLOOR



SECOND FLOOR PLAN
EQUIPMENT ROOM
CORRIDOR
RAISED FLR
LADDER
LEGEND
GRAPHIC SCALE
KEY PLAN & AREA DESIGNATION
C F BRAUN & CO
STC



MEZZANINE PLAN

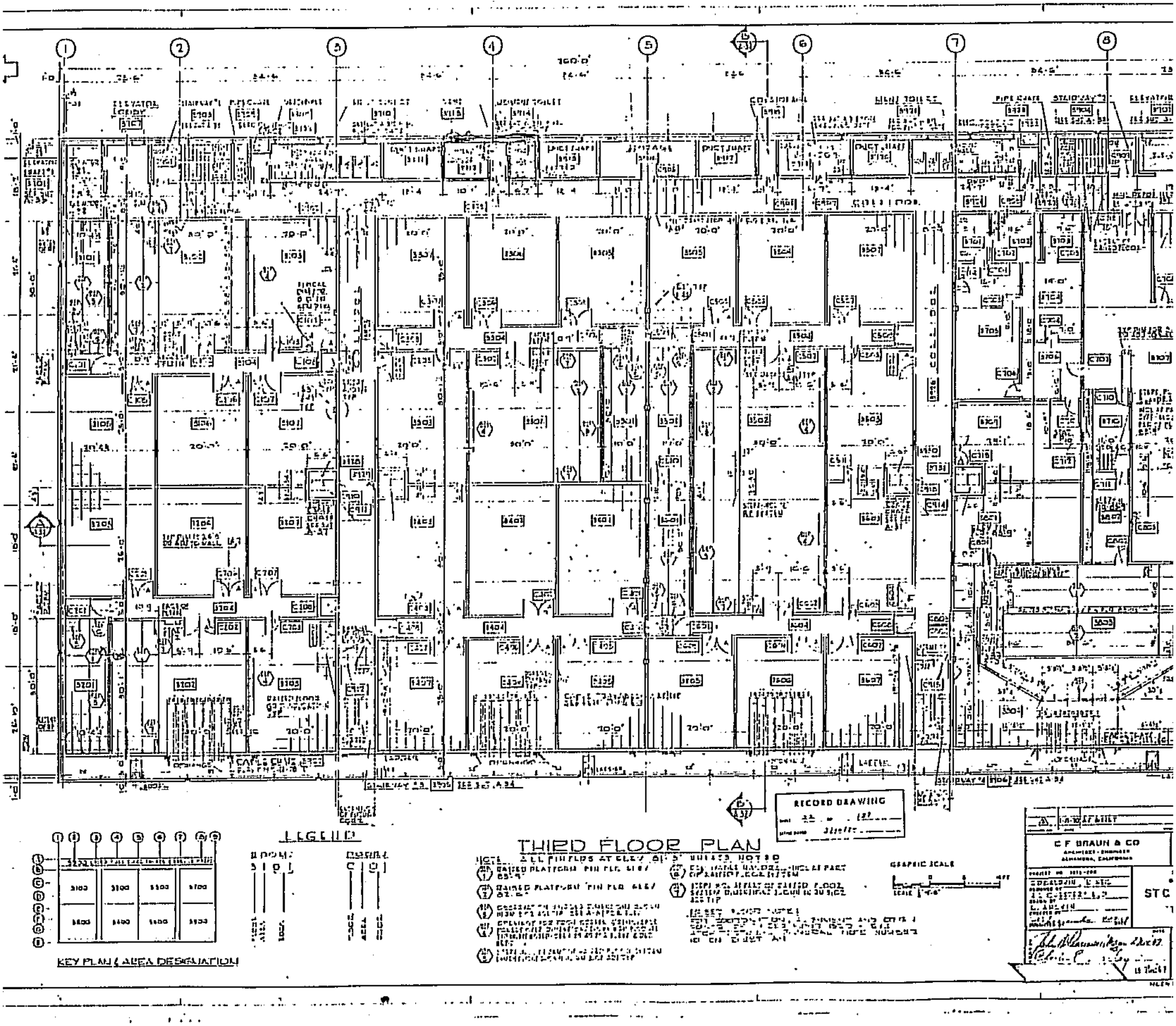
GRAPHIC SCALE

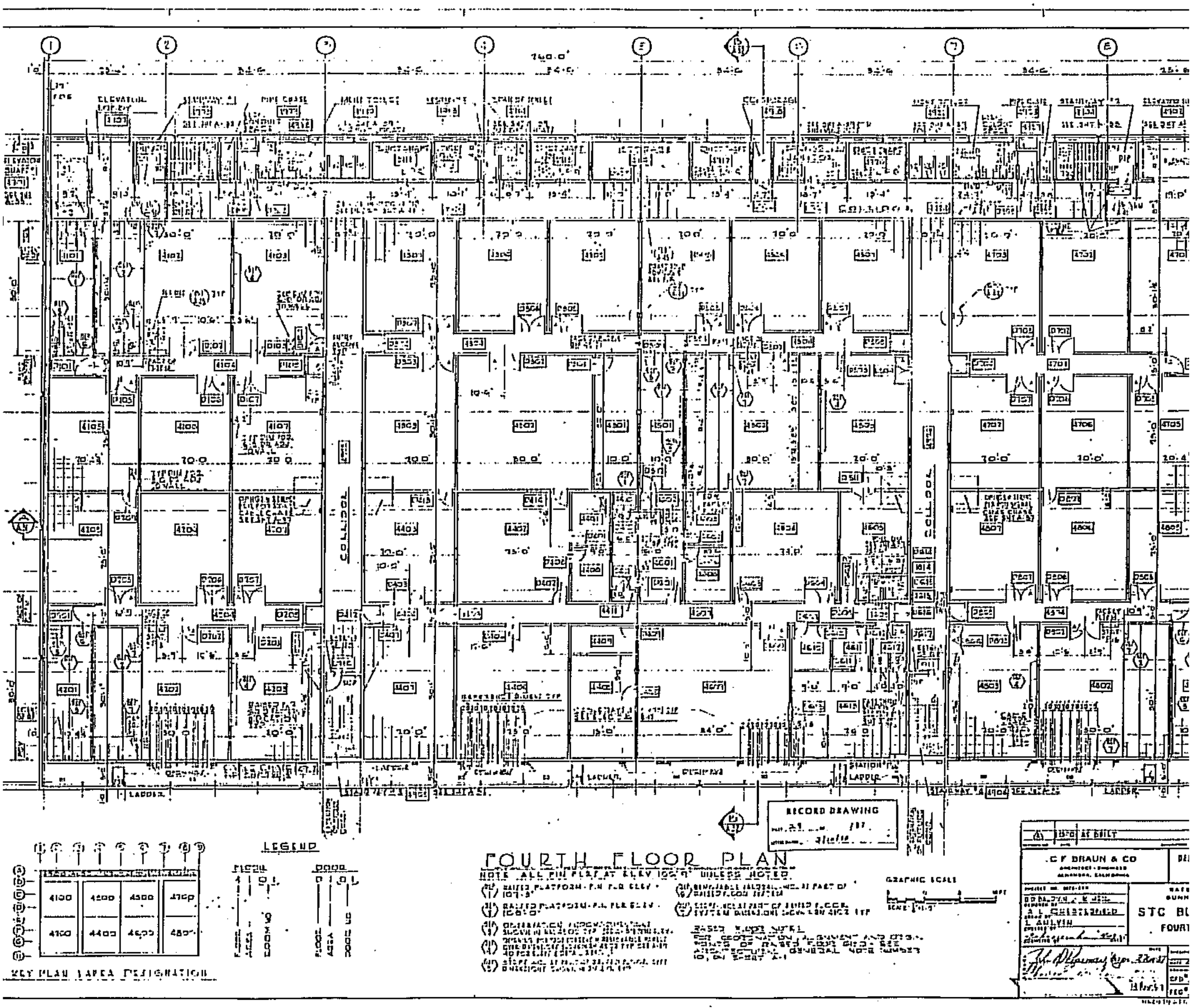
RECORD DRAWING

C F BRAUN & CO

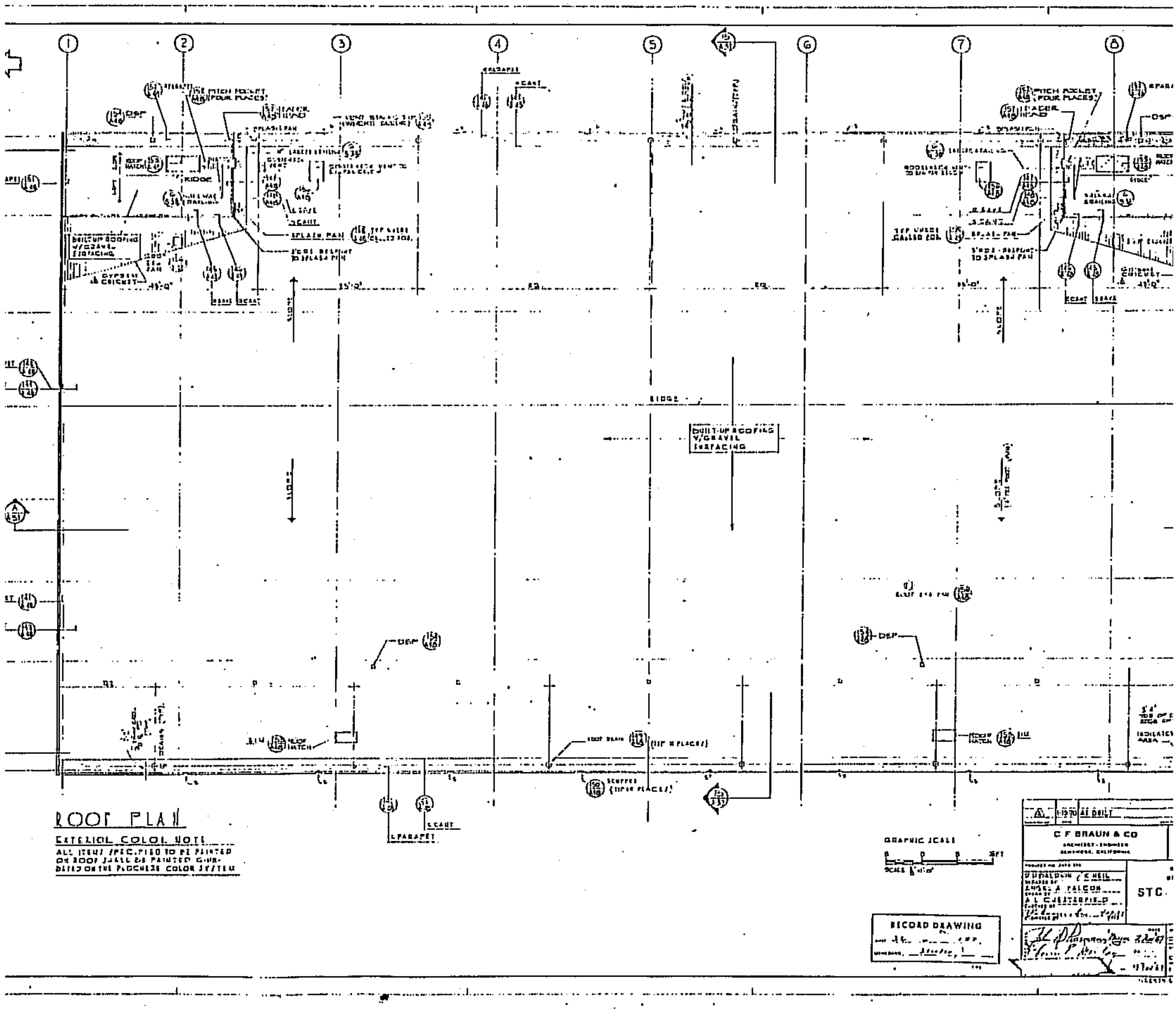
STC

HG7414

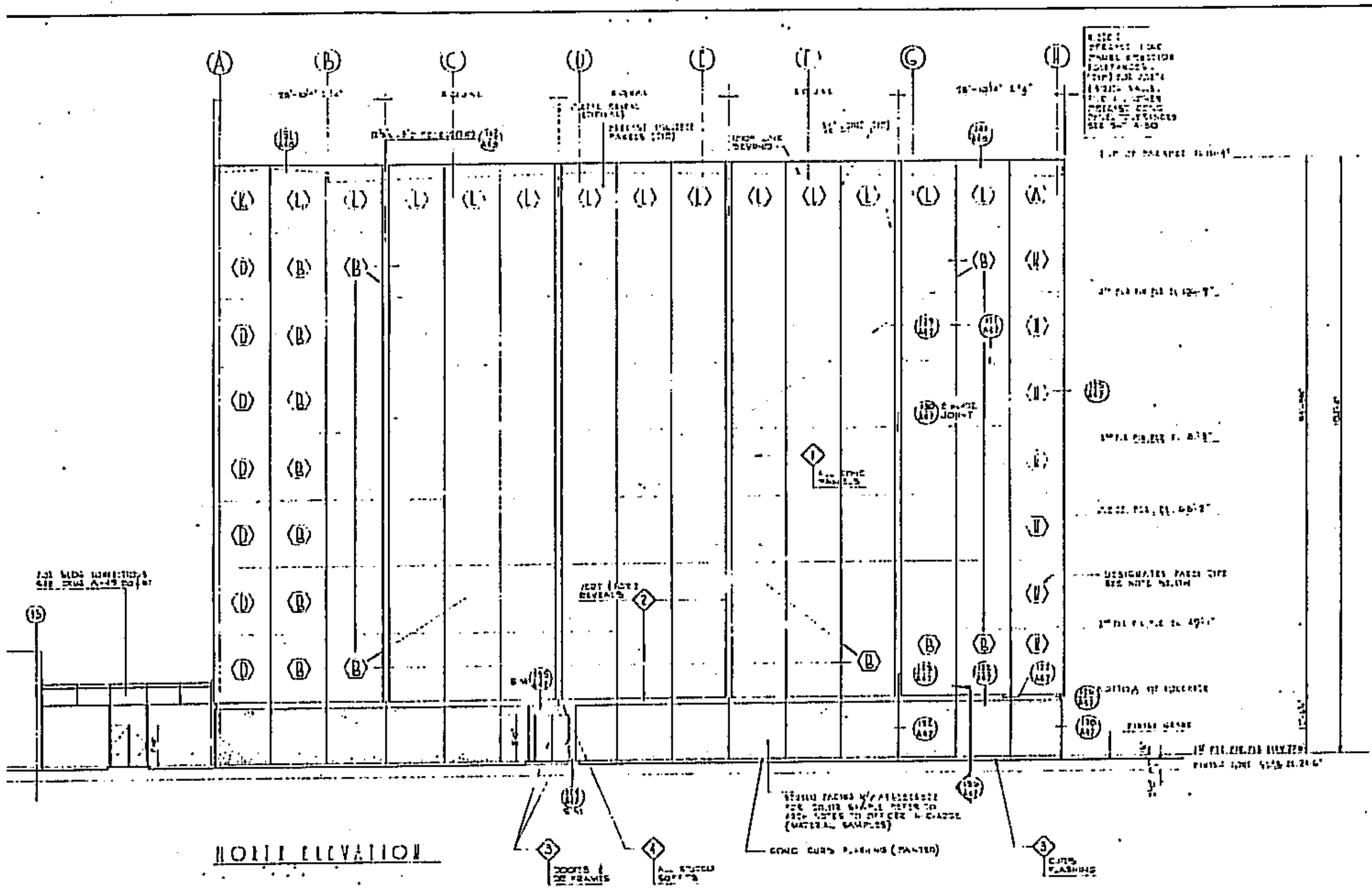

THIRD FLOOR PLAN
LEGEND
ROOM
DOOR
KEY PLAN & AREA DESIGNATION
RECORD DRAWING
GRAPHIC SCALE
C F BRAUN & CO
STC



FOURTH FLOOR PLAN
NOTE: ALL FIN FLR AT ELEV 100'-0" UNLESS NOTED
RECORD DRAWING
LEGEND
KEY PLAN AREA DESIGNATION
GRAPHIC SCALE
C F BRAUN & CO
STC BU
FOURT
CORRIDOR
ELEVATOR LOBBY
STAIRWAY #1
STAIRWAY #2
PIPE CHASE
MENS TOILET
LADDER
4100
4200
4300
4400
4500
4600
4700
4800



ROOF PLAN
EXTERIOR COLOR NOTE
ALL ITEMS SPECIFIED TO BE PAINTED ON ROOF SHALL BE PAINTED GUNBARREL ON THE PACHESE COLOR SYSTEM
BUILT-UP ROOFING W/GRAVEL SURFACING
RIDGE
SLOPE
GRAPHIC SCALE
C F BRAUN & CO
STC
RECORD DRAWING



NORTH ELEVATION

EXTERIOR COLOR LEGEND

SYMBOL	COLOR
1	G-24
2	BLACK
3	G-41
4	WHITE

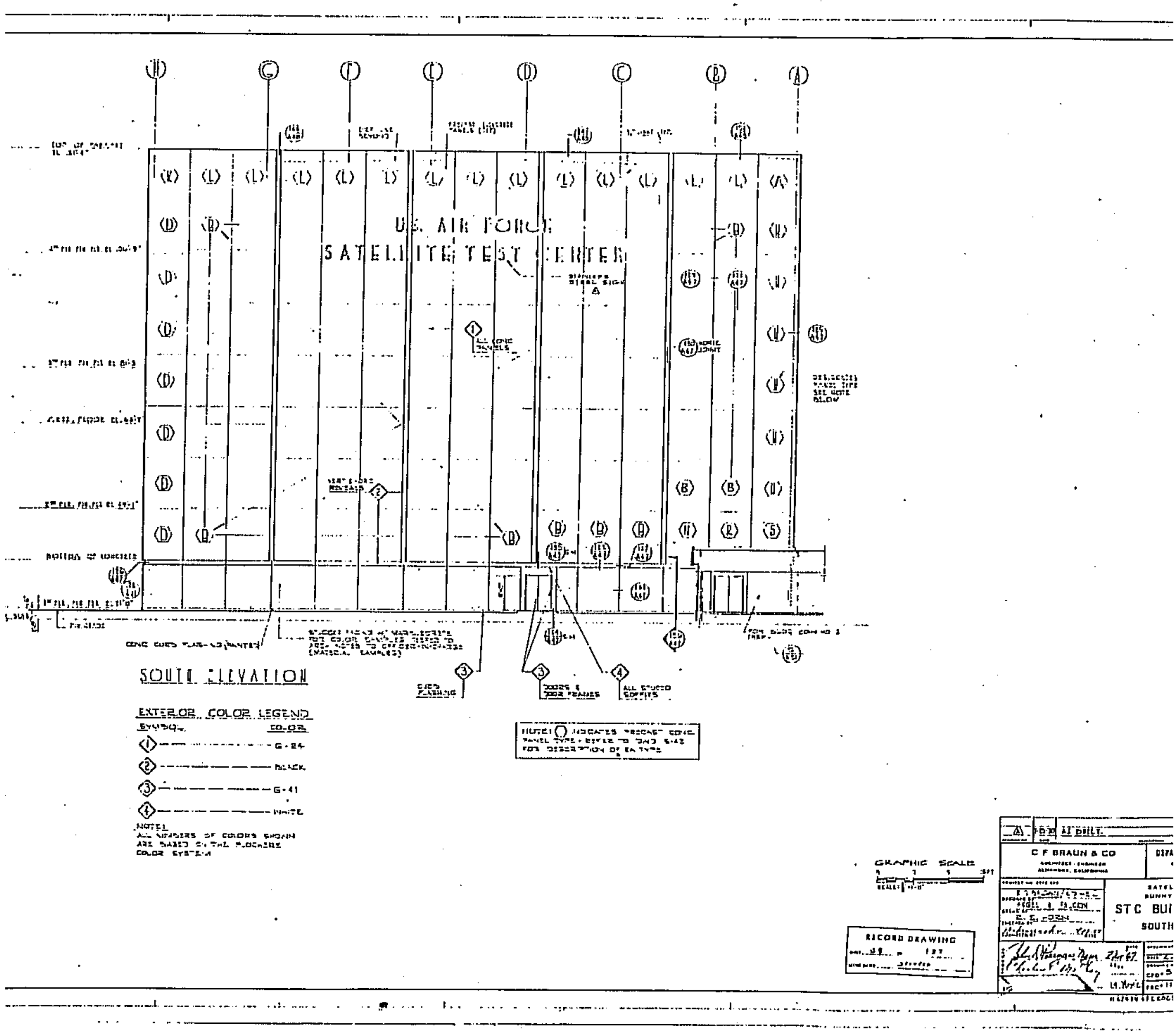
NOTE:
ALL NUMBERS OF COLORS SHOWN ARE BASED ON THE PLOCHERS COLOR SYSTEM

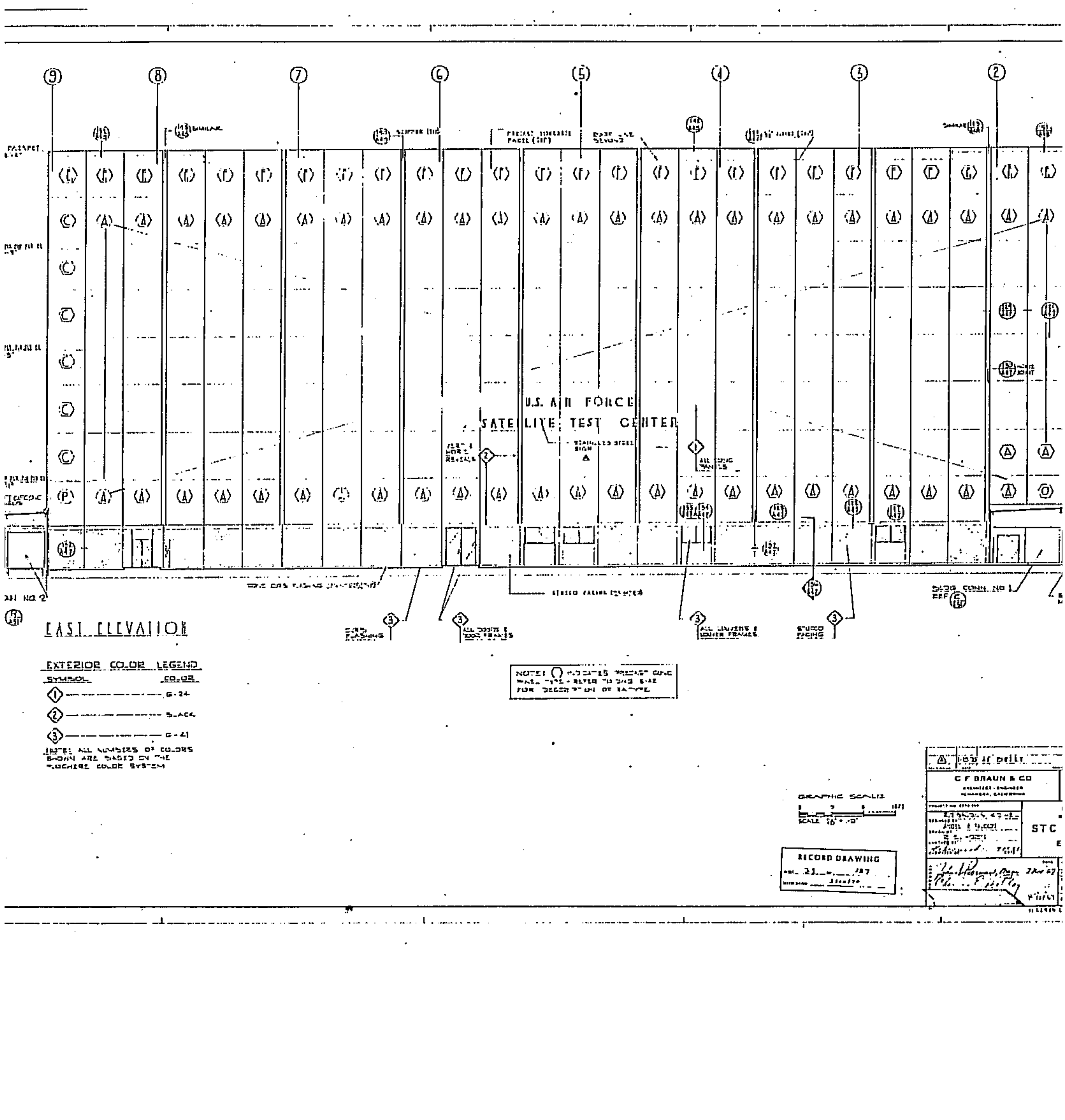
GRAPHIC SCALE

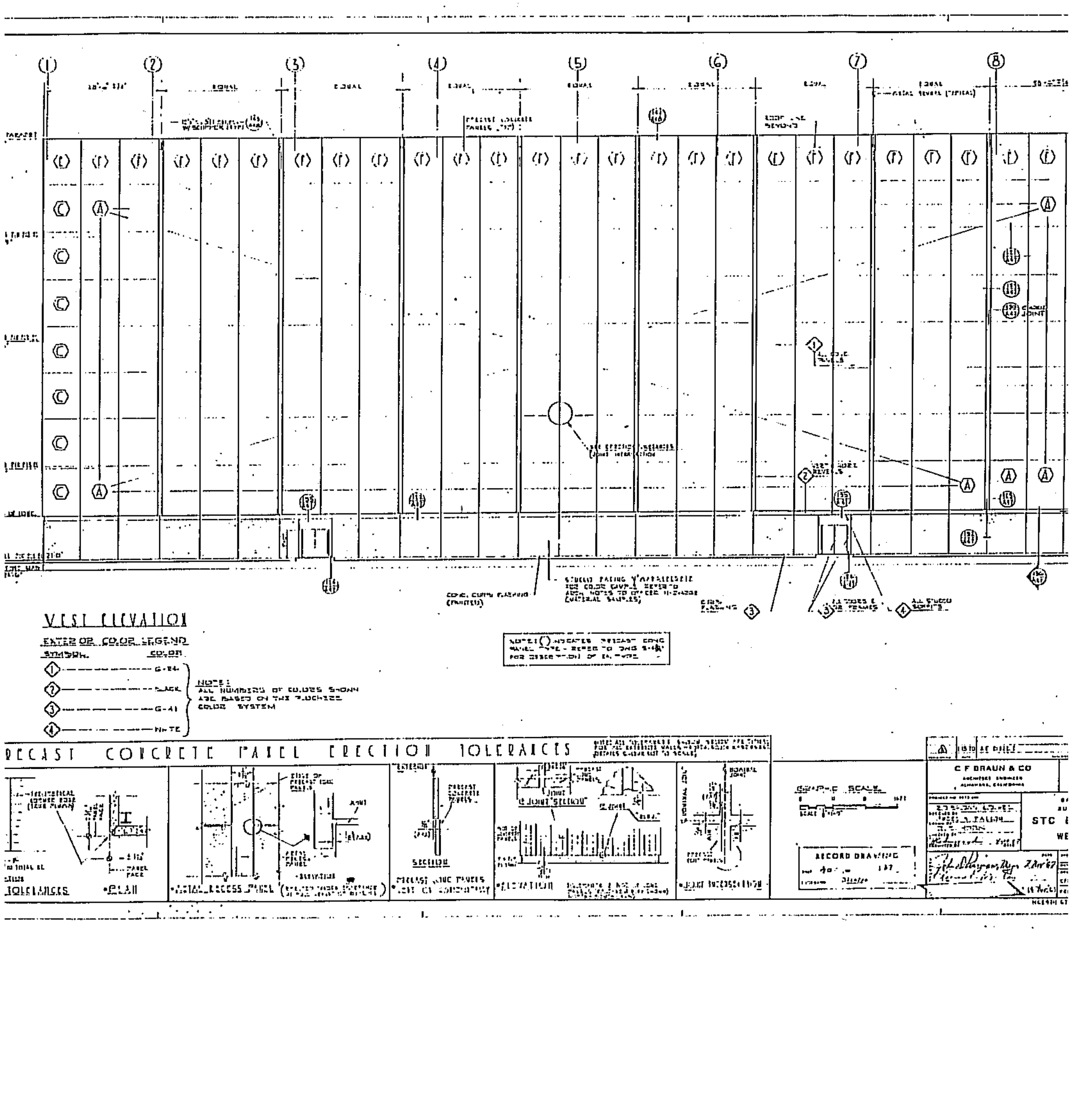
RECORD DRAWING

C F BRAUN & CO

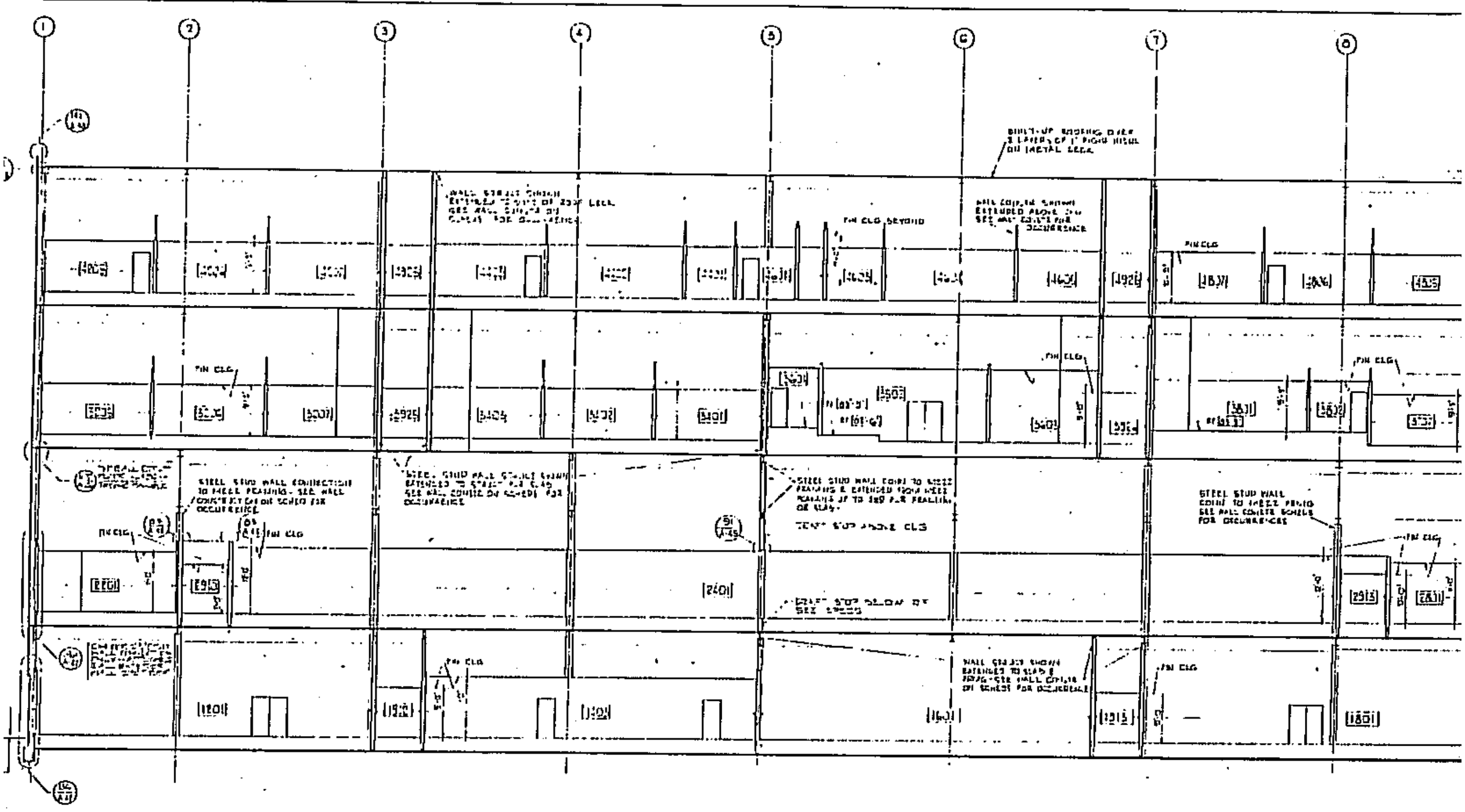
STC


U.S. AIR FORCE
SATELLITE TEST CENTER
SOUTH ELEVATION
EXTERIOR COLOR LEGEND
SYMBOL
COLOR
G-24
BLACK
G-41
WHITE
NOTE:
ALL NUMBERS OF COLORS SHOWN ARE BASED ON THE PLOCHERE COLOR SYSTEM
ALL CONC PANELS
ALL STUCCO SOFFITS
DOORS & DOOR FRAMES
GRAPHIC SCALE
RECORD DRAWING
AS BUILT
C F BRAUN & CO
STC BUI
SOUTH


U.S. AIR FORCE
SATELLITE TEST CENTER
EAST ELEVATION
EXTERIOR COLOR LEGEND
SYMBOL COLOR
1 G-24
2 BLACK
3 G-41
NOTE: ALL NUMBERS OF COLORS SHOWN ARE BASED ON THE PLOCHERE COLOR SYSTEM
ALL DOORS & DOOR FRAMES
ALL LOUVERS & LOUVER FRAMES
STUCCO FACING
GRAPHIC SCALES
RECORD DRAWING
C F BRAUN & CO
STC



WEST ELEVATION
EXTERIOR COLOR LEGEND
SYMBOL
COLOR
G-84
BLACK
G-41
WHITE
NOTE:
ALL NUMBERS OF COLORS SHOWN ARE BASED ON THE PLOCHERE COLOR SYSTEM
PRECAST CONCRETE PANEL ERECTION TOLERANCES
TOLERANCES
PLAN
SECTION
ELEVATION
JOINT INTERSECTION
C F BRAUN & CO
GRAPHIC SCALE
RECORD DRAWING
STC



LONGITUDINAL SECTION ..... A

GRAPHIC SCALE

RECORD DRAWING

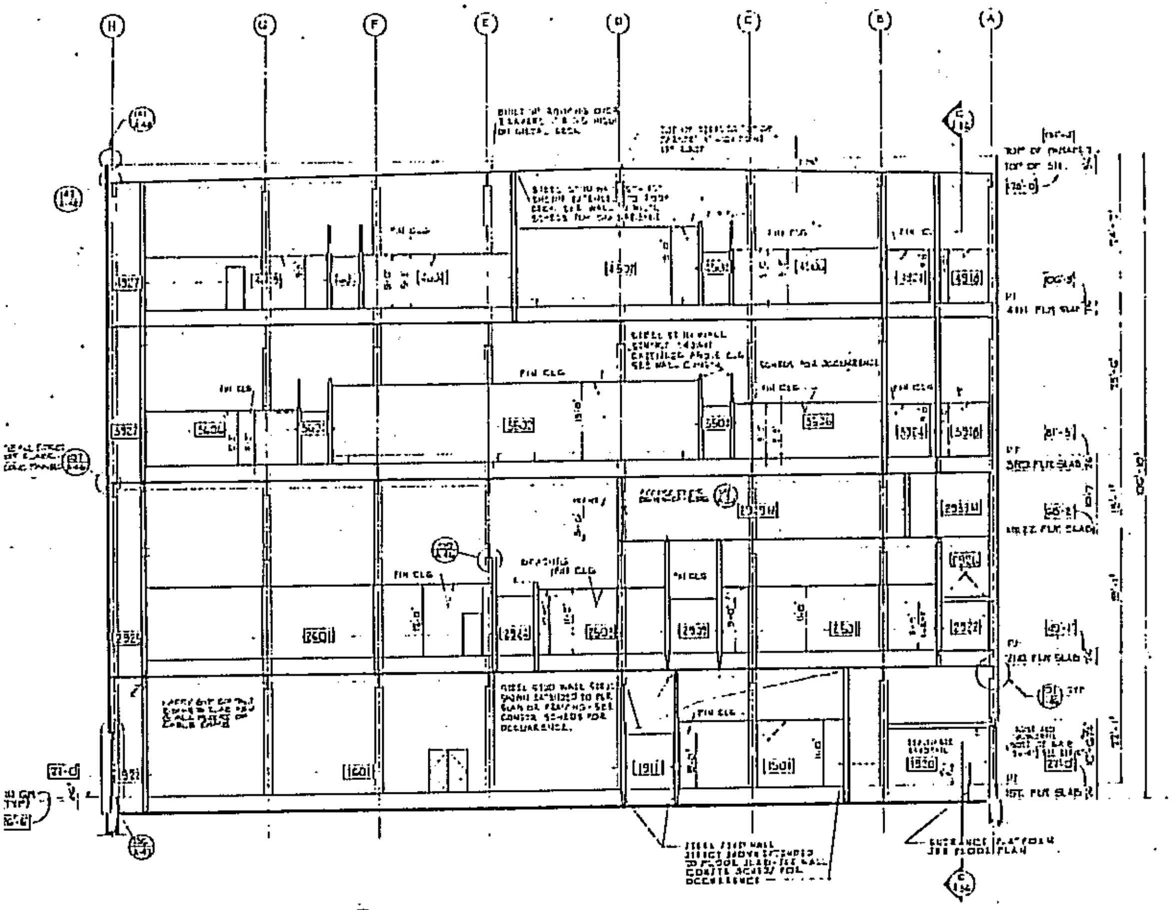
C F BRAUN & CO

ARCHITECT-ENGINEER

ALHAMBRA, CALIFORNIA

STC

BUI

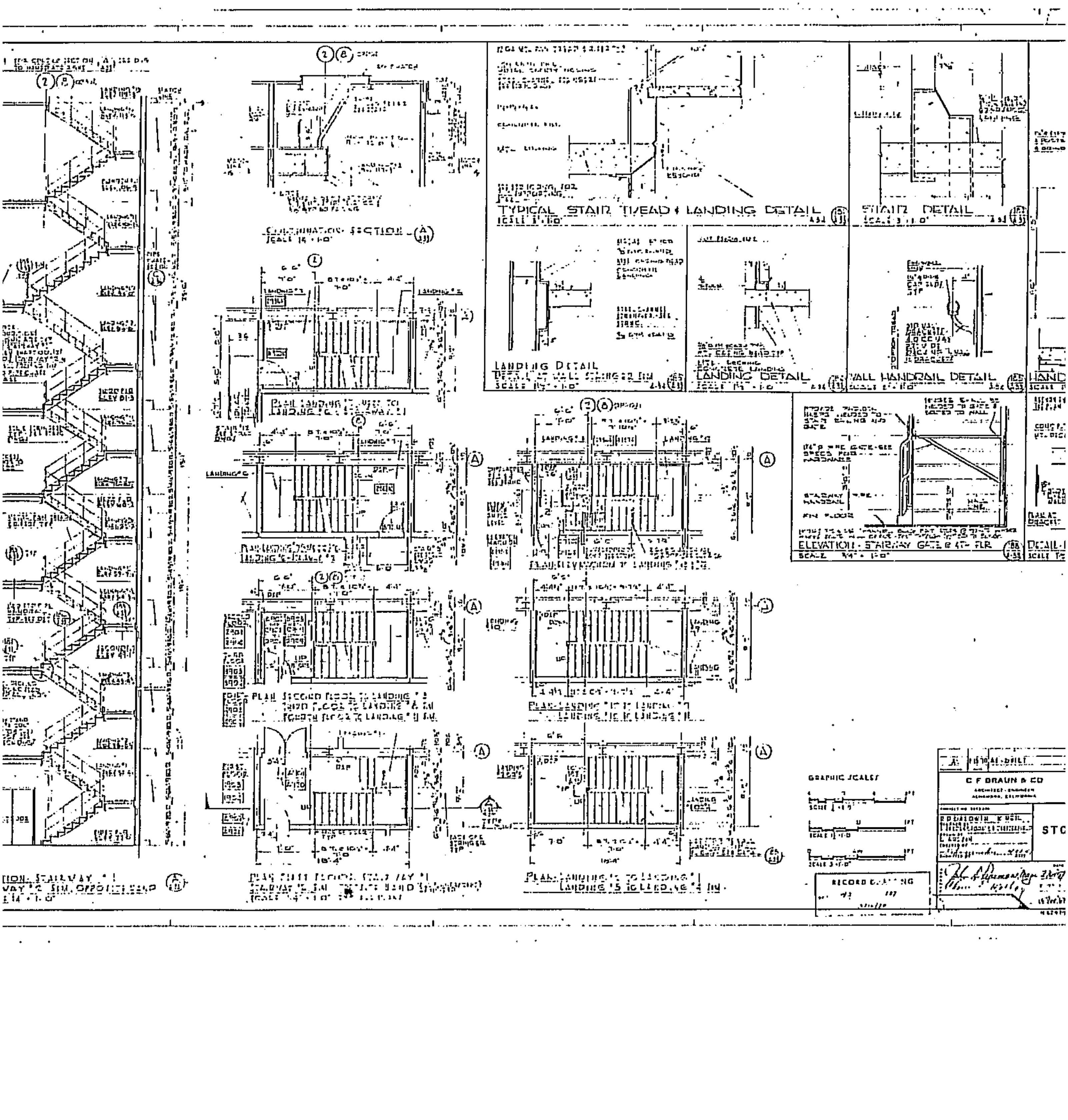


LATERAL SECTION

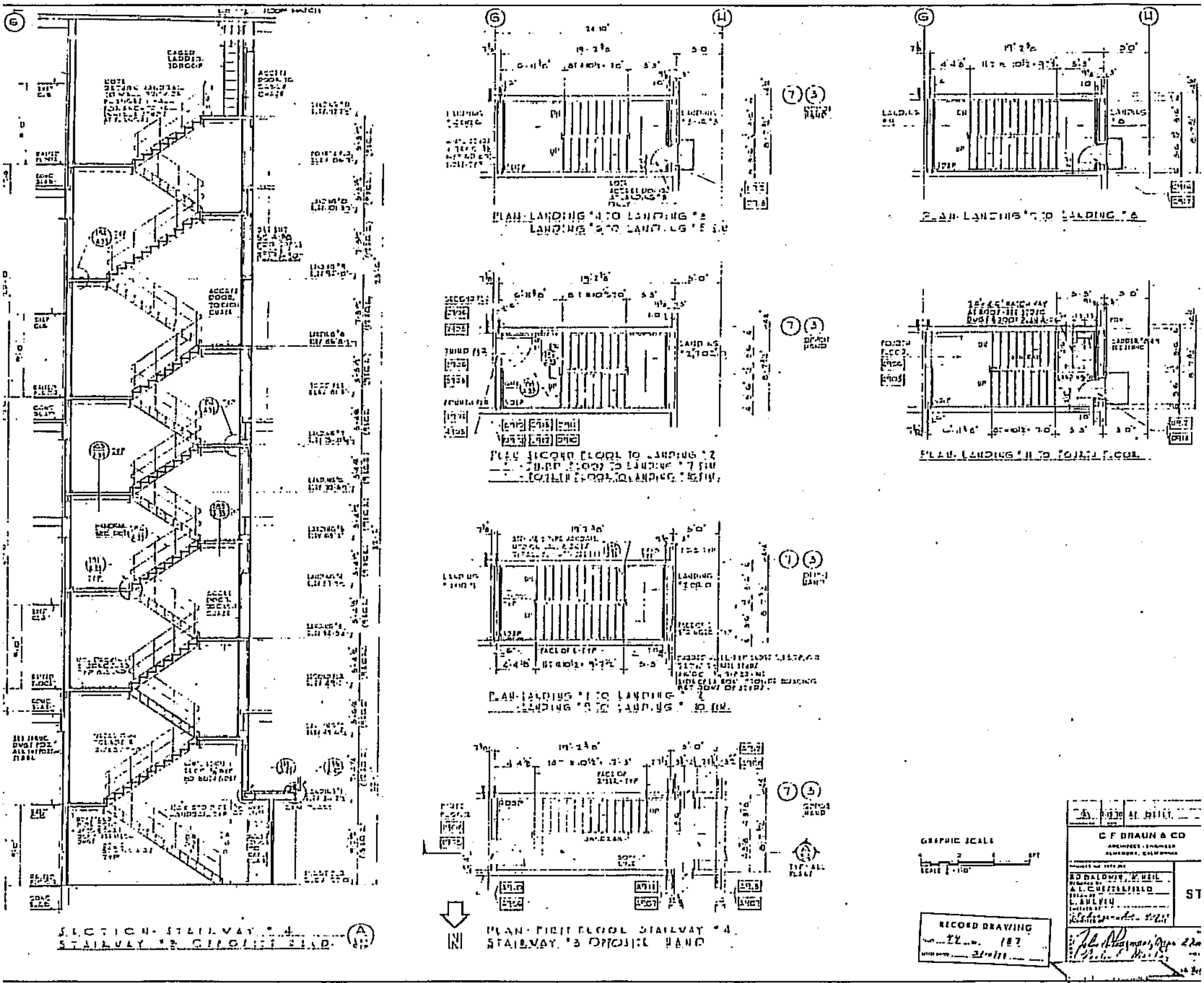
C F BRAUN & CO

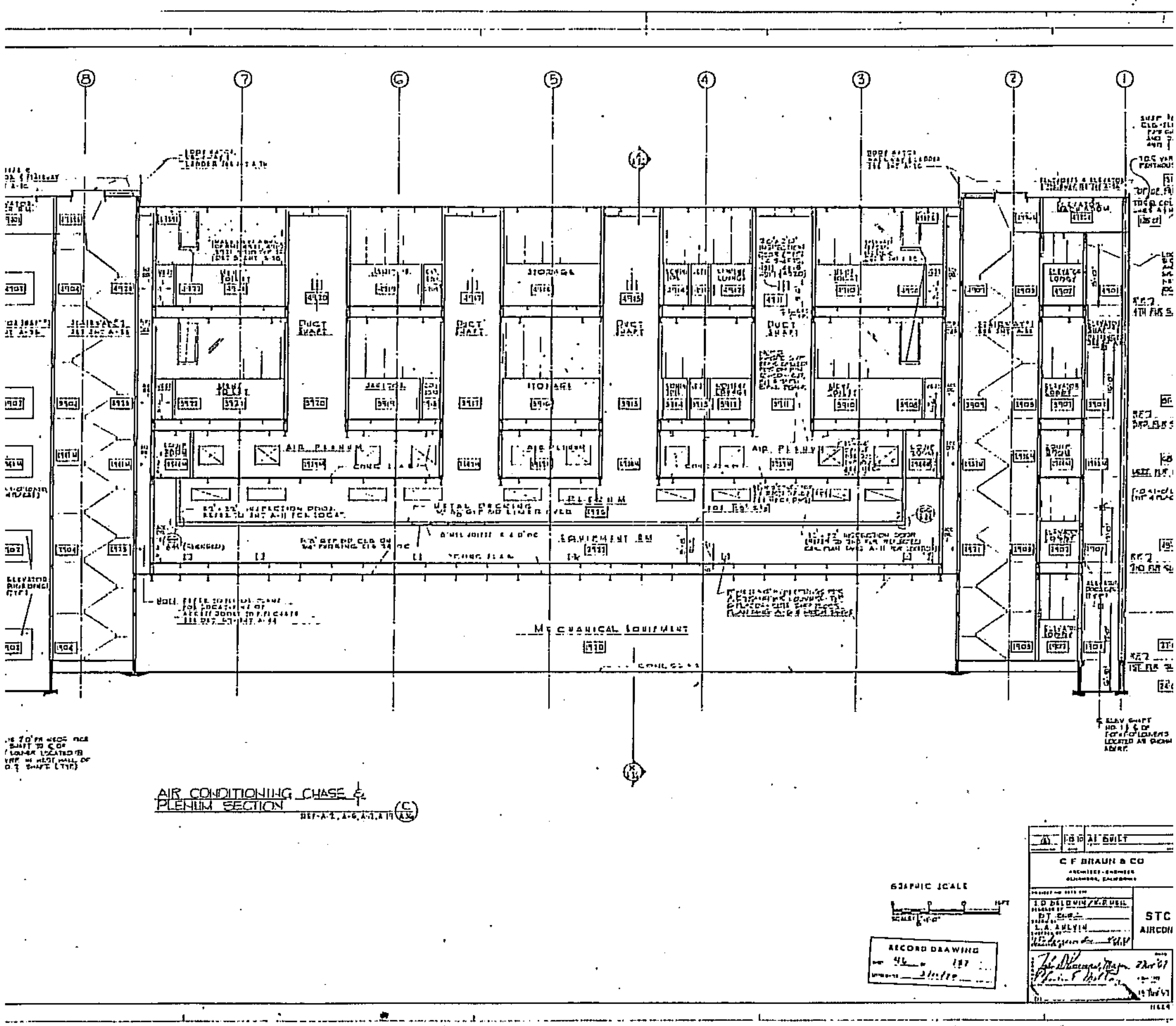
GRAPHIC SCALE

RECORD DRAWING

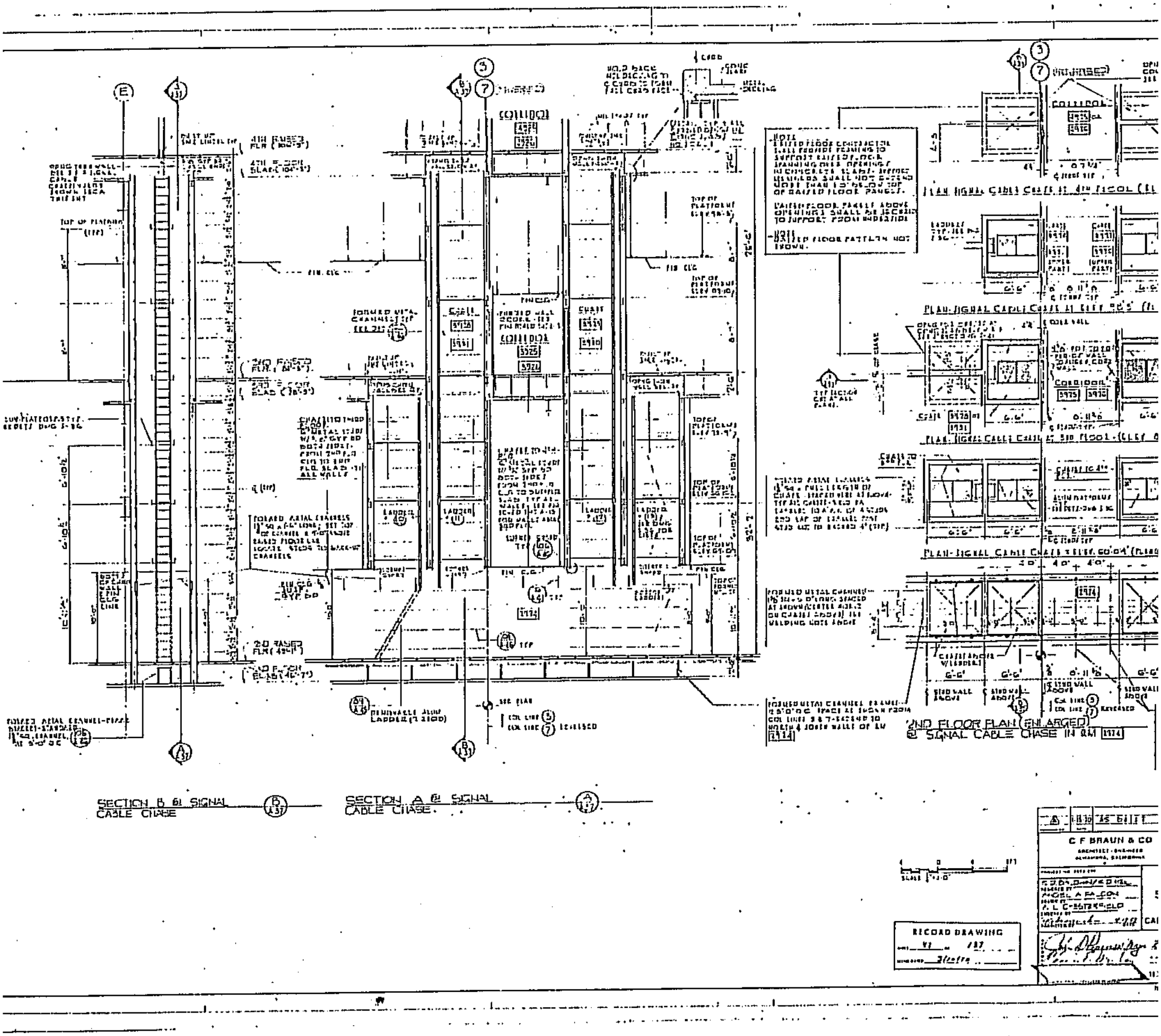


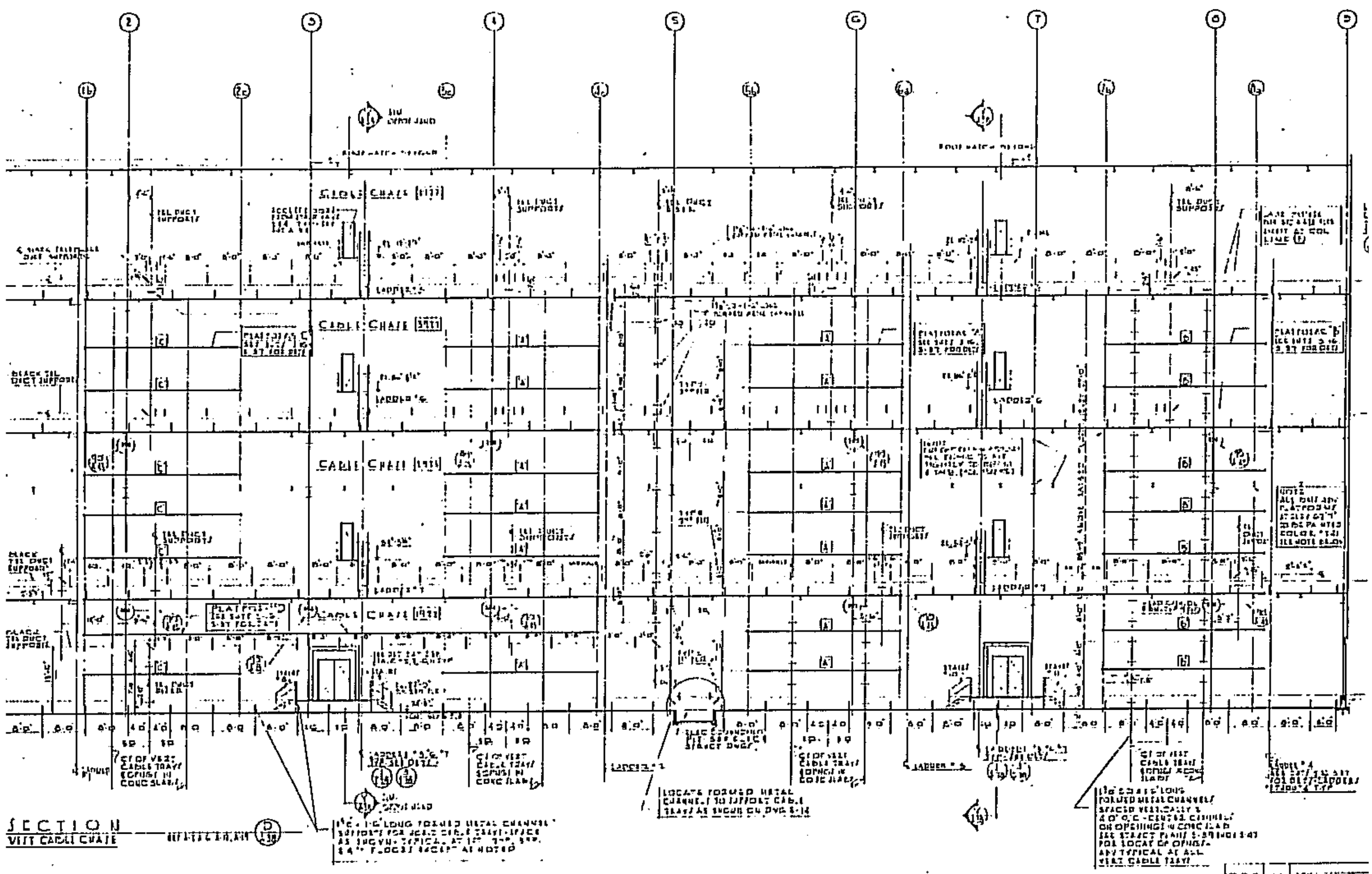
TYPICAL STAIR TREAD & LANDING DETAIL
STAIR DETAIL
LANDING DETAIL
LANDING DETAIL
WALL HANDRAIL DETAIL
ELEVATION - STAIRWAY GATE @ 4TH FLR.
CONTINUATION SECTION
PLAN - FIRST FLOOR - STAIRWAY #1
PLAN - SECOND FLOOR TO LANDING #3
PLAN - LANDING #5 TO LANDING #1
GRAPHIC SCALES
RECORD DRAWING
C F BRAUN & CO


SECTION - STAIRWAY #4
STAIRWAY #3 OPPOSITE HAND
PLAN - FIRST FLOOR STAIRWAY #4
STAIRWAY #3 OPPOSITE HAND
GRAPHIC SCALE
RECORD DRAWING
C F BRAUN & CO
ST



AIR CONDITIONING CHASE & PLENUM SECTION


SECTION B @ SIGNAL CABLE CHASE
SECTION A @ SIGNAL CABLE CHASE
2ND FLOOR PLAN (ENLARGED) @ SIGNAL CABLE CHASE IN RM 1114
C F BRAUN & CO
RECORD DRAWING



SECTION
VERT CABLE CHASE

NOTES:
1. REFER TO ELEC DWG E-14 FOR LAYOUT OF CABLE TRAYS & TEL DUCTS
2. LOCATIONS OF ALL FORMED METAL CHANNELS TO BE VERIFIED WITH AND CHECKED BY ELEC CONTRACTOR
3. STUDS TO BE LOCATED TO RECEIVE FORMED METAL CHANNELS AS SHOWN ON THIS DRAWING
4. ALL VERTICAL CHANNELS TO BE ATTACHED PER DETAIL ON SHT A-44
5. ALL HORIZONTAL OR DIAGONAL CHANNELS TO BE ATTACHED PER DETAIL SHT A-44

COLOR NOTE
ALL NUMBERS OF COLORS SHOWN ARE BASED ON THE FLOCHERE COLOR SYSTEM

GRAPHIC SCALE

RECORD DRAWING

C F BRAUN & CO
ARCHITECT - ENGINEER
ALHAMBRA, CALIFORNIA

Onizuka AFB, Calif. Monday, Feb. 10, 1992

Special Edition

Building 1003 undergoing repairs

Building 1003 is undergoing structural repairs as a result of recently-discovered damage caused by the 1989 Loma Prieta earthquake. The building is safe and there is no reason to vacate or halt normal operations. However, hazard notices have been posted to make people aware that the building may be susceptible to severe damage in the event of an earthquake of similar severity to the Loma Prieta quake.

Repairs to restore the building to its original standards should be completed by the end of February.

Damage was first discovered in October 1991 during renovations that exposed one of the building's vertical I-beams. Damage to a structural steel joint, called a lateral bracing joint, was found.

Two independent structural engineering firms were brought in to examine the damaged joint, inspect other visually accessible joints and provide expert assessments. Their assessment was that the damage was caused by the Loma Prieta earthquake and they identified 10 other joints that sustained lesser degrees of damage. They recommended to the Air Force that known damaged joints be repaired, all other lateral bracing joints be inspected and repaired as necessary, and

underway for repair of the 11 identified damaged joints. Repairs on these should be completed by mid-February. Concurrently, HQ AFSPACECOM contracted the Idaho National Engineering Laboratory (INEL) for an inspection of all perimeter structural joints in the building.

The INEL inspection was performed during the last week of January and the first week of February and involved cutting holes in interior walls at many locations to gain access to structural steel joints. In all, over 300 structural joints were inspected.

The INEL study was completed Feb. 7 and disclosed an additional 22 joints that sustained some level of damage and require repairs. This new information resulted in accelerating the work schedules to complete the additional repairs as quickly as possible.

The repairs are being done in order of the severity of the damage to the joints involved. Eleven of the 33 joints identified have been repaired and repair to the joints needed to bring the building back to its original standards will be completed by the end of February. Additional repairs will continue beyond that time to joints that sustained minor damage, but they are inconsequential to the overall structural integrity of the building.

Building 1003 repair nearly complete

Progress continues to the structural repairs of Building 1003 as nearly two-thirds of the work has been completed towards bringing the building back to its original standards, increasing to 94 percent by Thursday.

The repairs are due to recently-discovered damage caused by the 1989 Loma Prieta earthquake. Base officials remind people that the building is safe and there is no reason to vacate or halt normal operations. Hazard notices have been posted to make people aware that the building may be susceptible to severe damage in the event of an earthquake of similar severity to the Loma Prieta quake, however, the notices should be removed by the end of the week.

Repairs to completely restore the building to its original standards should be completed by the end of February.

Damage was first discovered in October 1991 during renovations that exposed one of the building's vertical I-beams. A damaged structural steel joint, called a lateral bracing joint, was found.

Since then, two independent structural engineering firms and the Idaho National Engineering Laboratory have examined the building's joints. Over 300 joints were inspected and 33 were identified as sustaining some level of damage.

The repairs are being done in order of the severity of the damage to the joints. Twenty of the 33 joints identified have been repaired with 10 more due to be completed by Thursday. Additional repairs will continue beyond Thursday, but they are inconsequential to the overall structural integrity of the building.

After repairs have been completed, an in-depth structural engineering analysis of the entire building design will be conducted.

Sgt. Shenndoe Williams of Personnel was the big winner at the "Apollo Night" talent show, held Feb. 7 at the NASA Ames Theater on Moffett Field. Williams took the $100 first prize after a sing-off against the vocal group One Idea. Williams then donated half of the prize money to Onizuka's Afro-American Heritage Committee. Ten acts took part in the talent show, and an audience of about 150 people turned out for the event. Photo by Capt. Art Haubold.

Read their lips: This is not a tax cut

IRS lowers income tax withholding rates...but you may pay more

Thanks to the Internal Revenue Service, you may take home bigger paychecks this year. But you may also have to pay more when it comes time to file your income taxes for 1992.

The IRS has issued a new edition of **Publication 15, Circular E, "Employer's Tax Guide,"** to all employers. The publication contains new withholding tables for federal income tax.

These tables will reduce the amount of income tax withheld for most low- and middle-income service members and federal employees, thereby increasing their take-home pay. People who have their taxes withheld at the married rate may see as much as $345 more in their take-home pay over the next year. Those withheld at the single rate may see up to $172 more.

THIS IS NOT A TAX CUT. Service members and federal employees' tax liabilities will remain the same. Only the amount withheld will change.

As a result of these lower withholding rates, some people who received a refund from their 1991 taxes may receive a smaller refund from their 1992 tax withheld. Others who received refunds in 1991 may owe money to the IRS when they file their 1992 taxes. Employees who usually owe when they file may find they owe more be affected by these new tables. People who withheld at the married rate with wages subject to withholding of $90,200 or more will see no change.

Workers who withhold at the single rate will also see no change if their wages subject to withholding are $53,200 or more. Wages subject to withholding are total annual wages reduced by $2,300 for each withholding allowance claimed.

People who don't want their withholding changed should complete a new Form W-4 and submit it to their finance office. The IRS says that people may claim the same number of withholding allowances as before, but should indicate on Line 6 of the W-4 that they want additional taxes withheld each payday.

For people withheld at the married rate, the amount on Line 6 should be $345 divided by the number of paydays in the year — 26 for civilians, 12 for military. People withheld at the single rate should use $172 on Line 6. These amounts should be added to any amount already shown on Line 6. Another option is to reduce the number of allowances claimed on the W-4.

More information on determining the correct withholding can be

Table 1: Scheme Costs for Life Safety Only
(Cost in Million Dollars)

	SCHEME NUMBER						
Description	1	2*	3	4	5	6	7
Mobilization	$2.40	$2.00	$1.80	$2.15	$1.80	$2.65	$0.2
Structural Upgrade	$8.90	$7.10	$6.40	$5.95	$7.50	$5.95	$2.5
Security	$1.70	$1.00	$0.90	$1.50	$0.90	$1.90	$0.8
Operations	$30.00	$1.00	$1.00	$1.00	$1.00	$1.00	$40.0
Total	$43.00	$11.10	$10.10	$10.60	$11.20	$11.50	$43.6

*Scheme 2 cost includes .5M for shaketable tests

Table 2: Itemized Structural Cost for Life Safety Only
(Cost in Million Dollars)

	SCHEME NUMBER						
Description	1	2*	3	4	5	6	7
Structural Frame	$1.80	$3.80	$3.20	$1.50	$4.30	$2.10	$0.80
Foundations	$0.20	$0.20	$0.20	$0.20	$0.20	$0.10	$0.80
Diaphragms	$2.90			$1.15			$0.65
Cladding	$3.80	$2.90	$2.80	$2.80	$2.80	$3.45	
Misc. Arch. & Equip.	$0.20	$0.20	$0.20	$0.30	$0.20	$0.30	$0.30
Total	$8.90	$7.10	$6.40	$5.95	$7.50	$5.95	$2.55

Cost Data from Reference 14 - Onizuka Air Force Base, Building 1003, Seismic Study, Cygna Group, March 31, 1993.

Table 3: Scheme Additional Costs for Operation Prote
(Cost in Million Dollars)

	SCHEME NUMBER						
Description	1	2	3.	4	5	6	
Mobilization		$0.15	$0.15	$0.15	$0.15	$0.10	$(
Structural Upgrade		$1.25	$1.25	$2.00	$1.25	$1.50	$:
Security		$0.20	$0.20	$0.45	$0.20	$0.30	$(
Operations				$60.00		$40.00	$4(
Total		$1.60	$1.60	$62.60	$1.60	$41.90	$4:

Table 4: Itemized Structural Cost for Operation Protec
(Cost in Million Dollars)

	SCHEME NUMBER						
Description	1	2	3	4	5	6	'
Structural Frame		$0.50	$0.50	$0.50	$0.50	$0.50	$0
Foundations							
Diaphragms		$0.65	$0.65	$1.20	$0.65	$0.80	$1
Cladding							
Misc. Arch. & Equip.		$0.10	$0.10	$0.30	$0.10	$0.20	$0
Total		$1.25	$1.25	$2.00	$1.25	$1.50	$2.

Cost Data from Reference 14 - Onizuka Air Force Base, Building 1003, Seismic Study, Cygna Group, March 31, 1993.

APPENDIX C

BLDG. 1003
PHOTOGRAPHS, EVALUATION, SEISMIC ANALYSIS, DEFICIENCIES AND COST

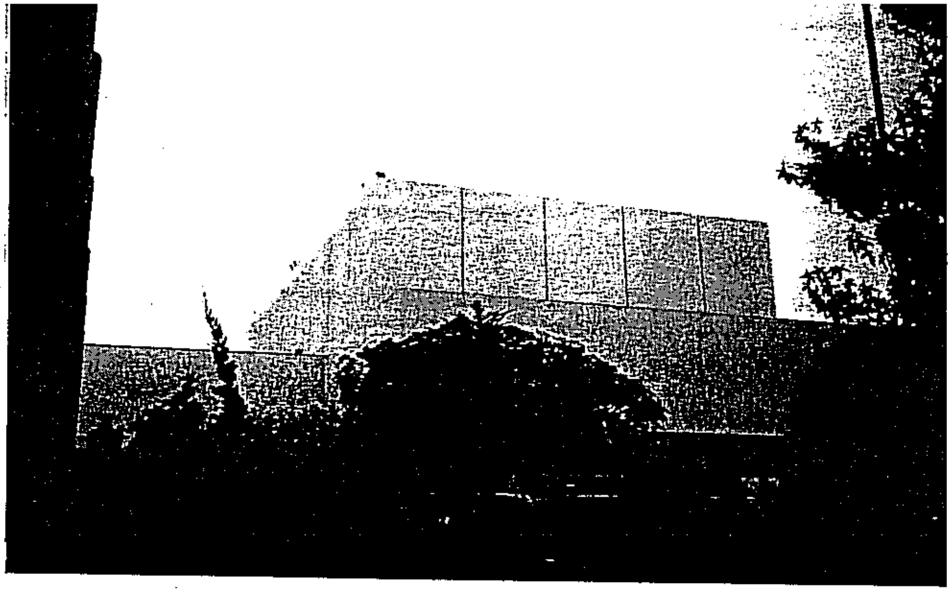


Photo 1. North view of Building 1003 behind, surrounded by Buildings 1001 and 10031 in foreground.

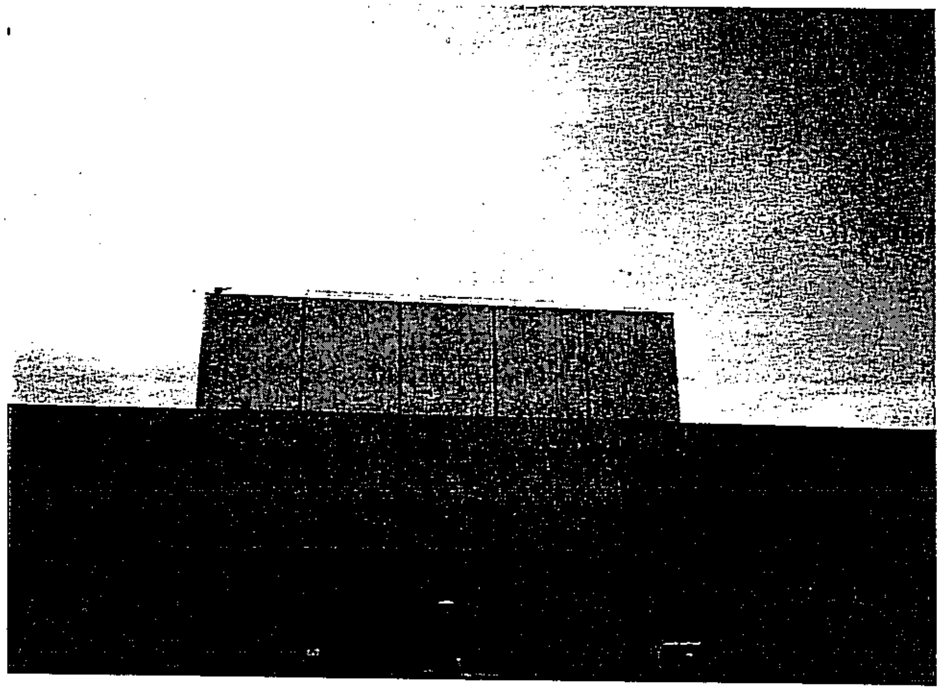


Photo 3. Northwest view of Building 1003 behind, surrounded by Buildings 10031 and 10032 in foreground.

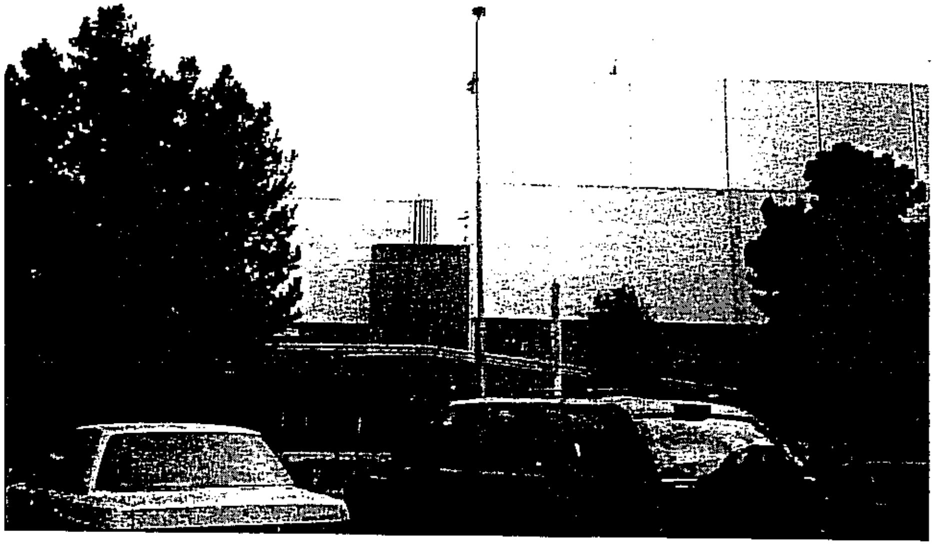
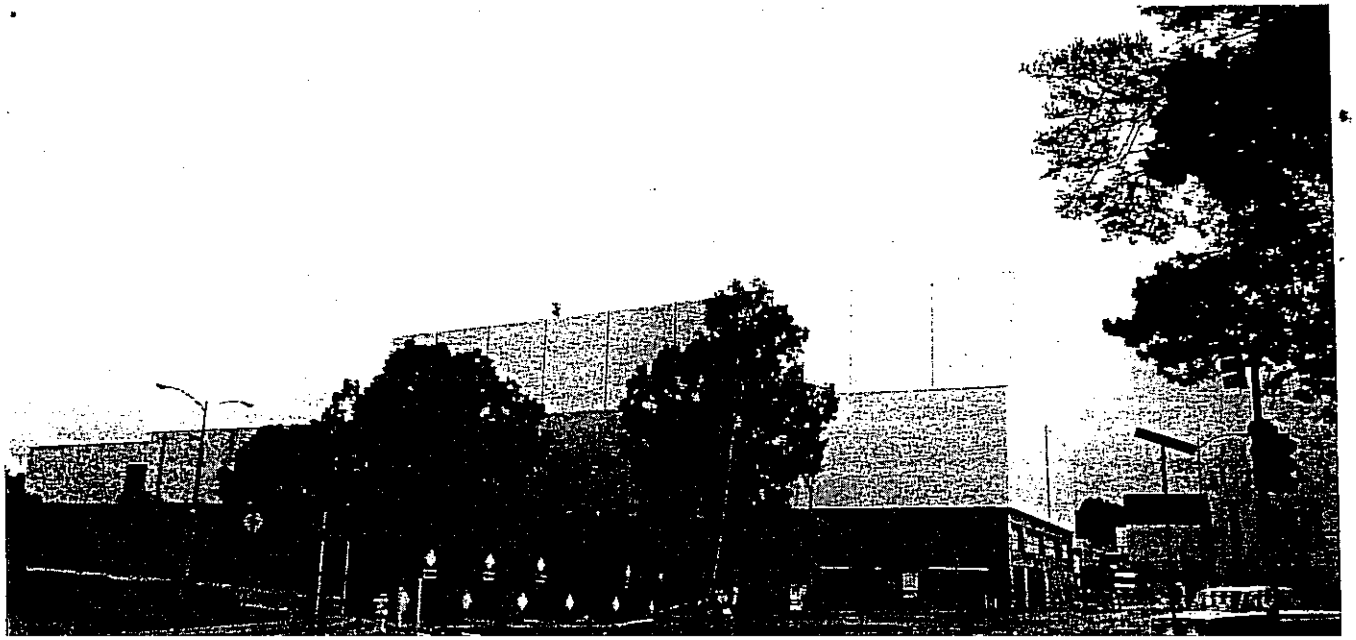


Photo 5. West view of Building 1003 behind, surrounded by Building 10032 in foreground.



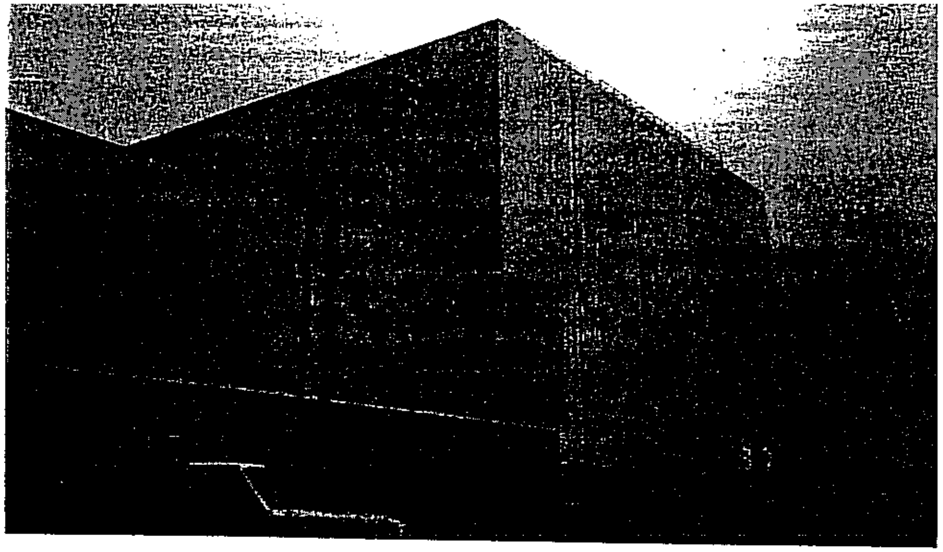
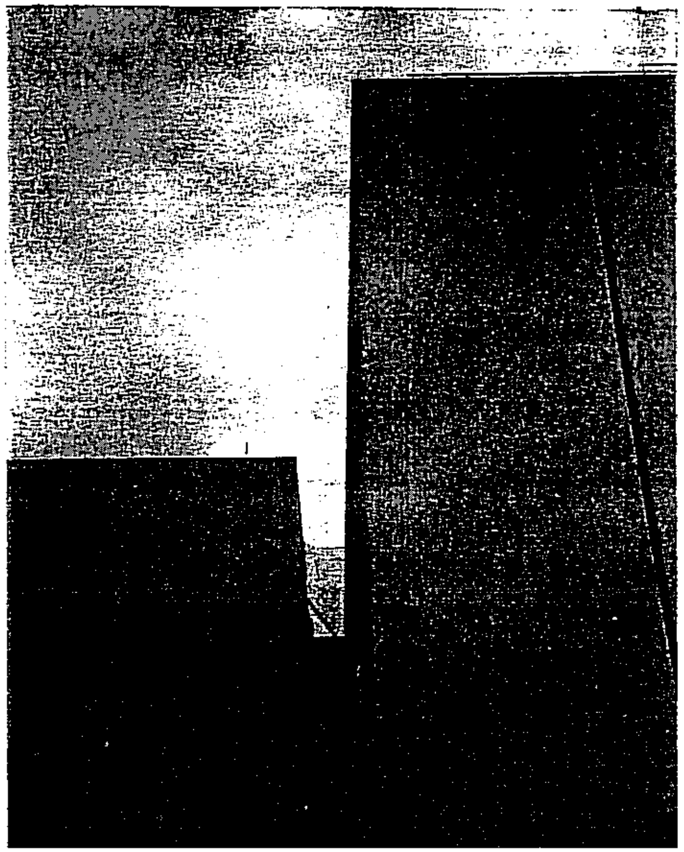


Photo 7. South view of Building 1003 behind, and Building 10032 in foreground.



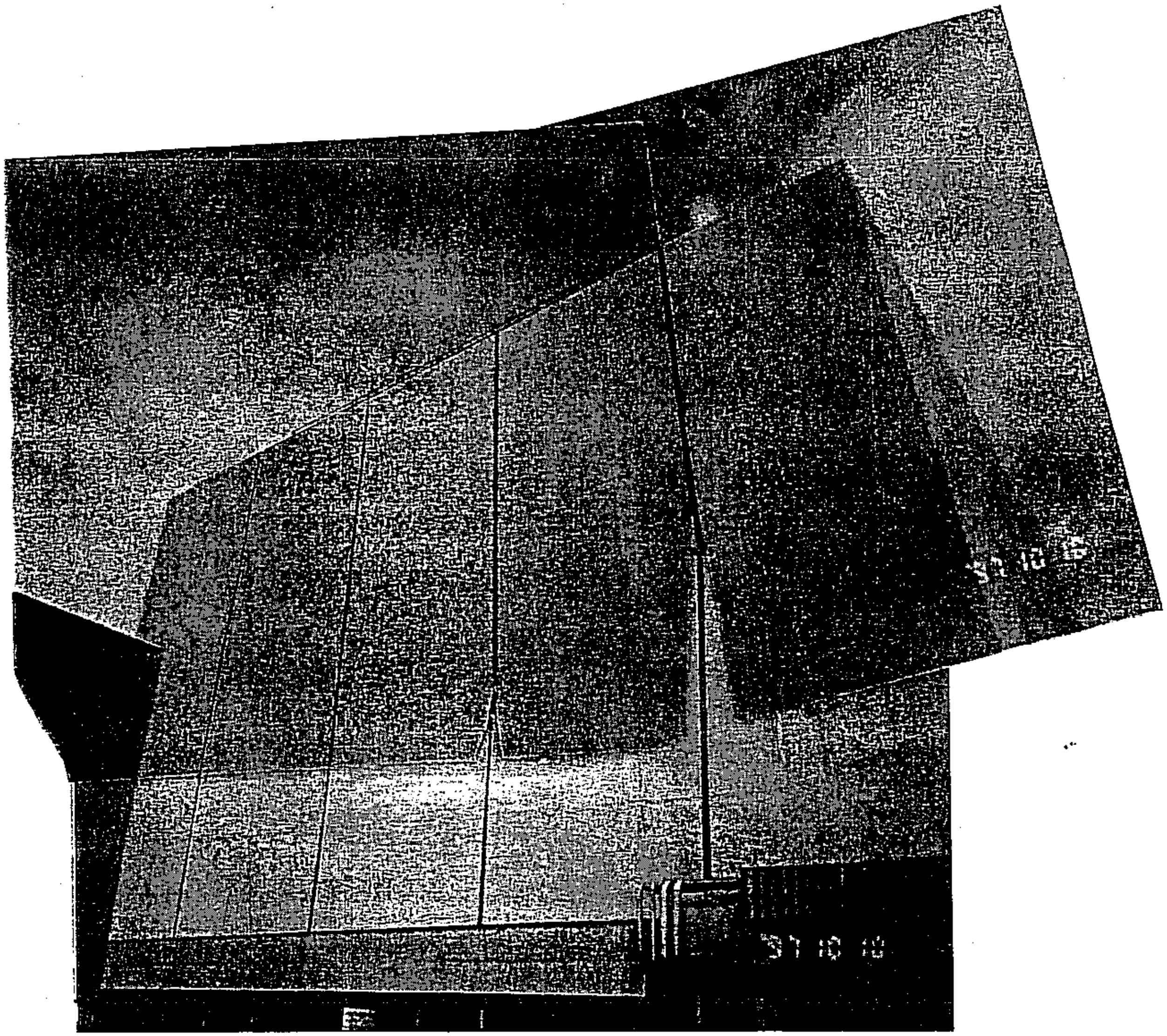


Photo 9. South close-up view of Building 1003.

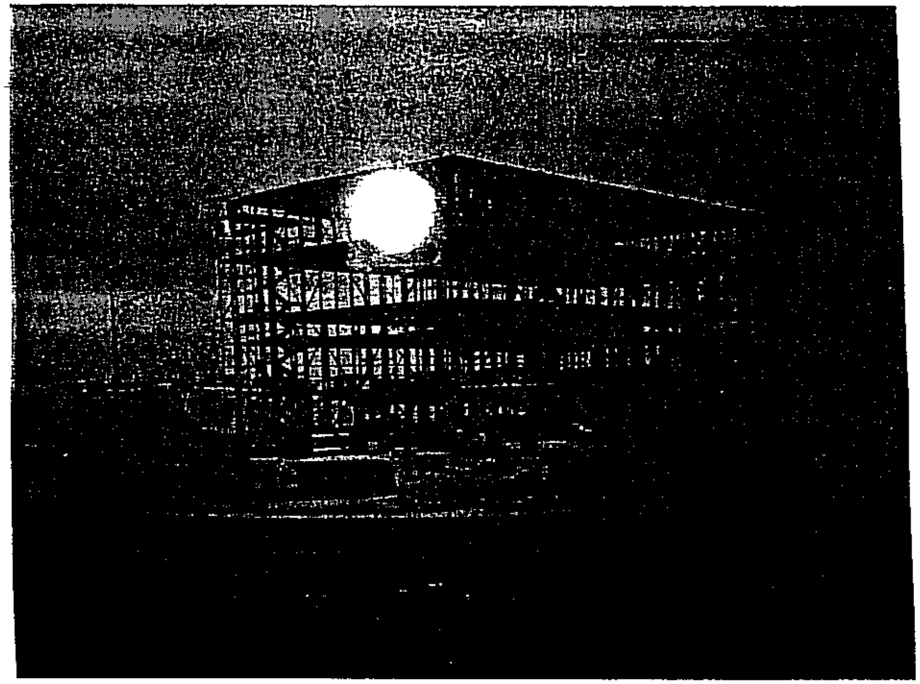


Photo 10. Existing photograph of the construction of Building 1003 as displayed in Lobby. Notice the framing of four story levels with 19 to 25 feet story heights.

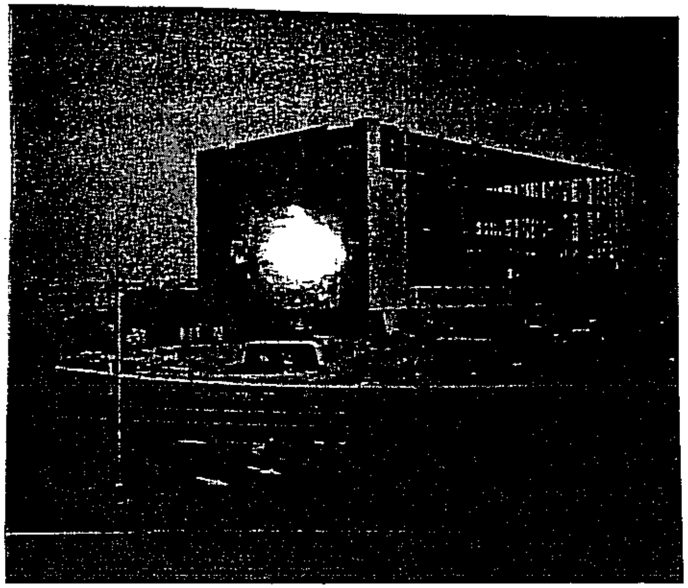
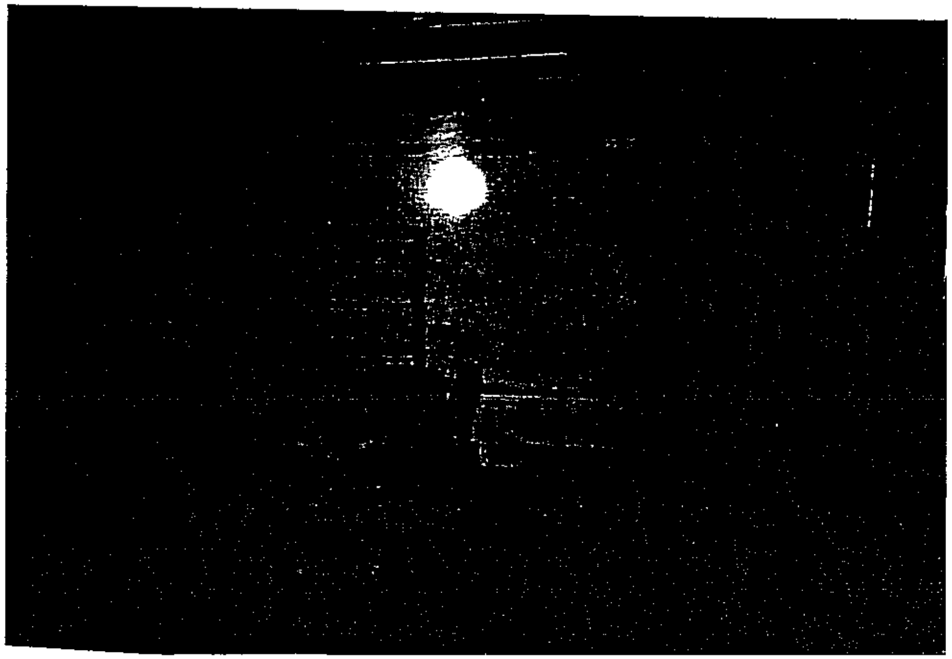


Photo 11. Existing photograph of the construction of Building 1003 as displayed in Lobby. Notice the

Photo 12. Existing photograph of the failure of a gusset plate following the 1989 Loma Prieta Earthquake. The failure is located in the cracked plate at the joining of a diagonal brace to the column. This was discovered accidentally during the reconstruction of a third floor bathroom. The crack was repaired but there is no information on whether a study was performed to inspect and evaluate all other similar connections in the building.



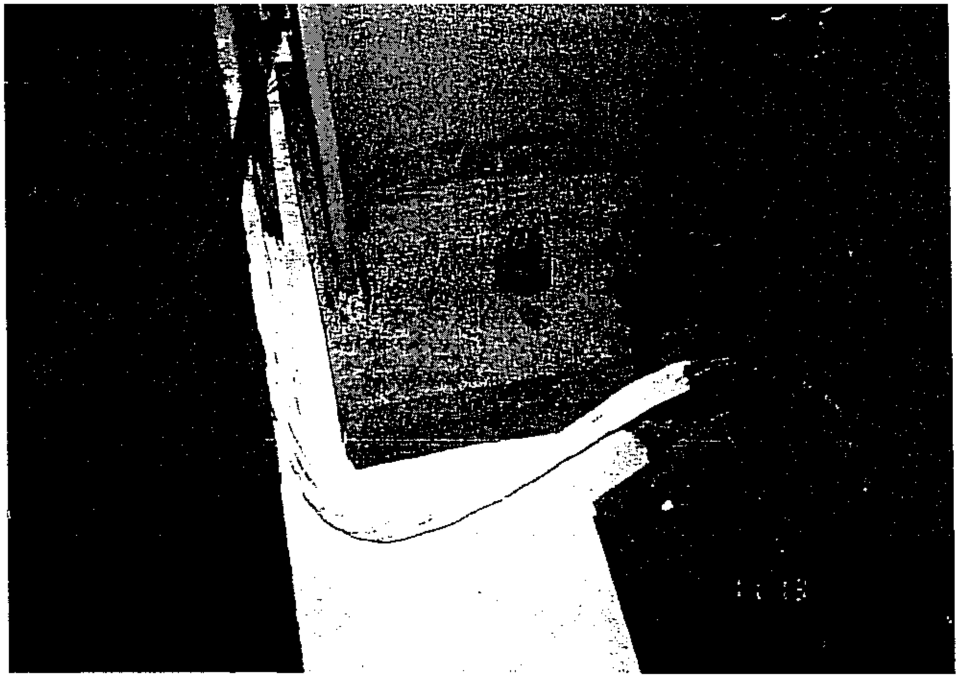
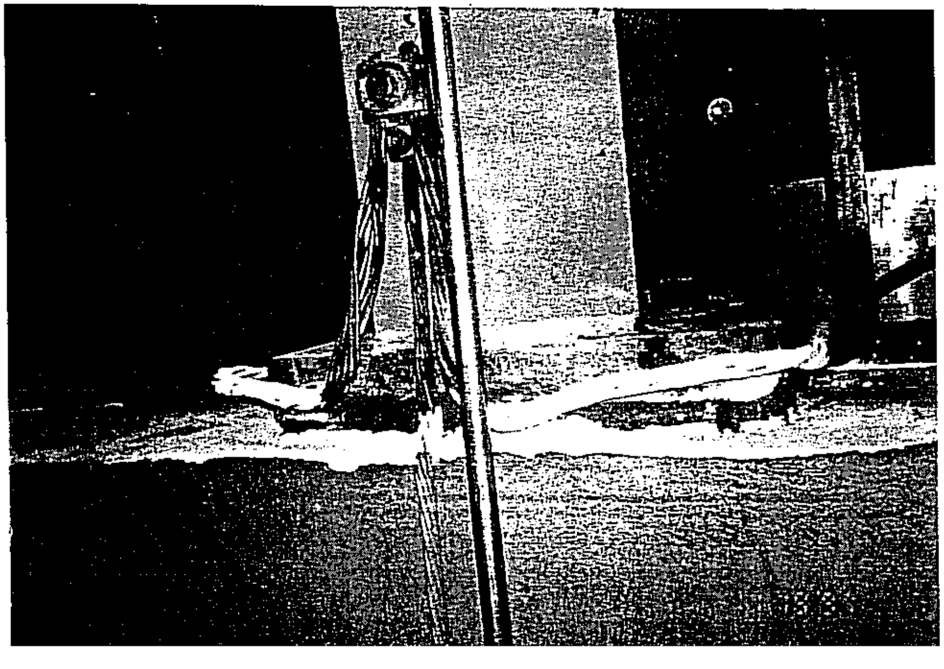


Photo 14. Typical column base anchorage to grade beam as seen on the ground floor mechanical room. Notice the connection and bolt installation are not designed as a moment connection.



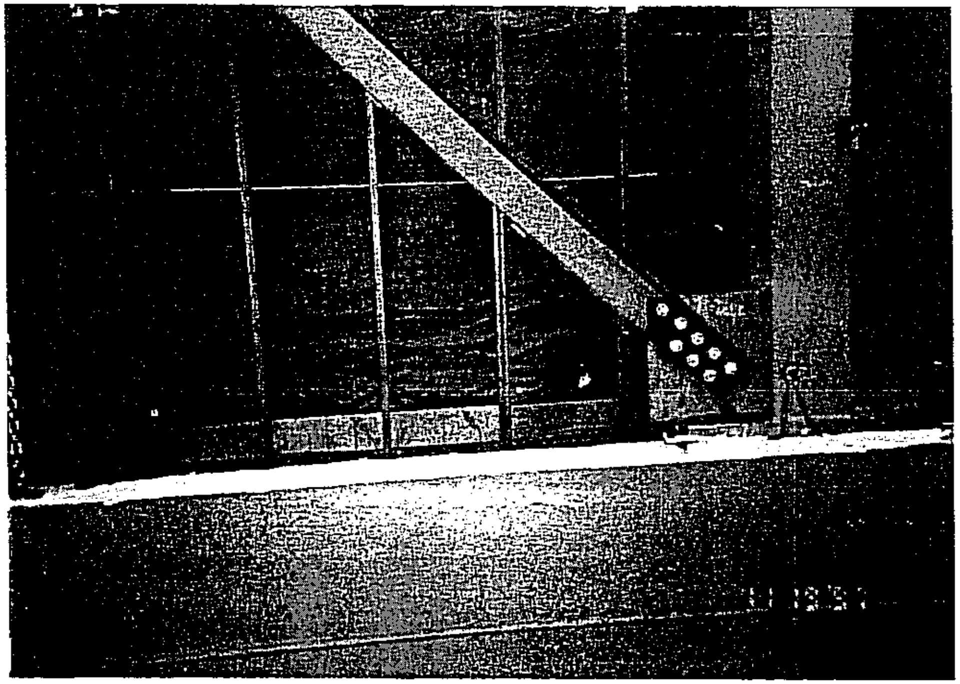
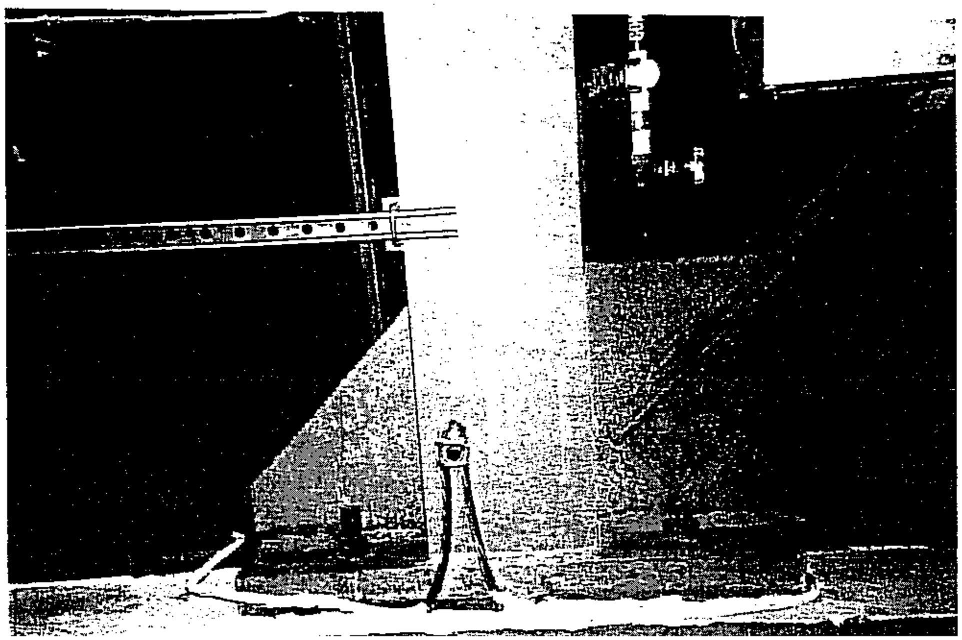


Photo 16. Typical diagonal bracing along building perimeter for lateral shear.



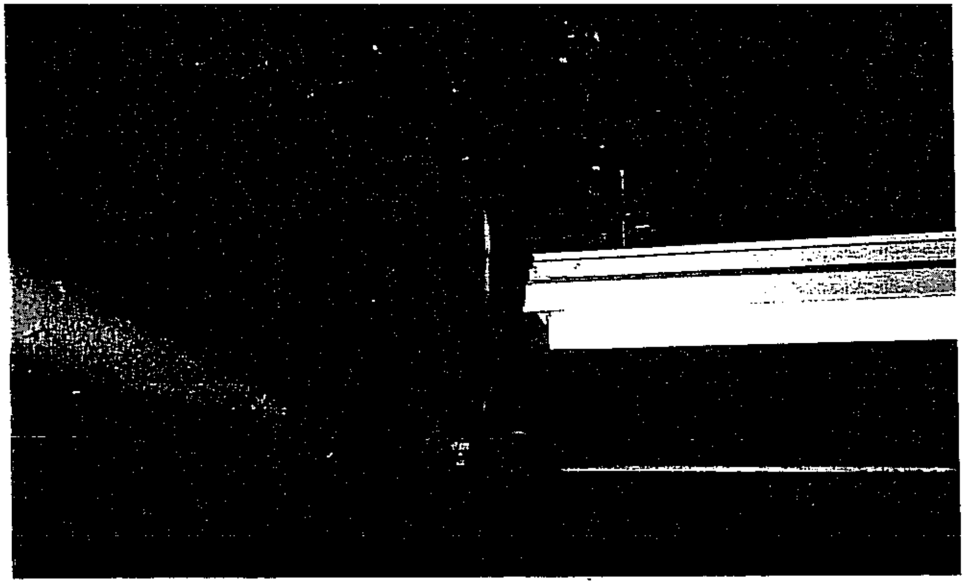
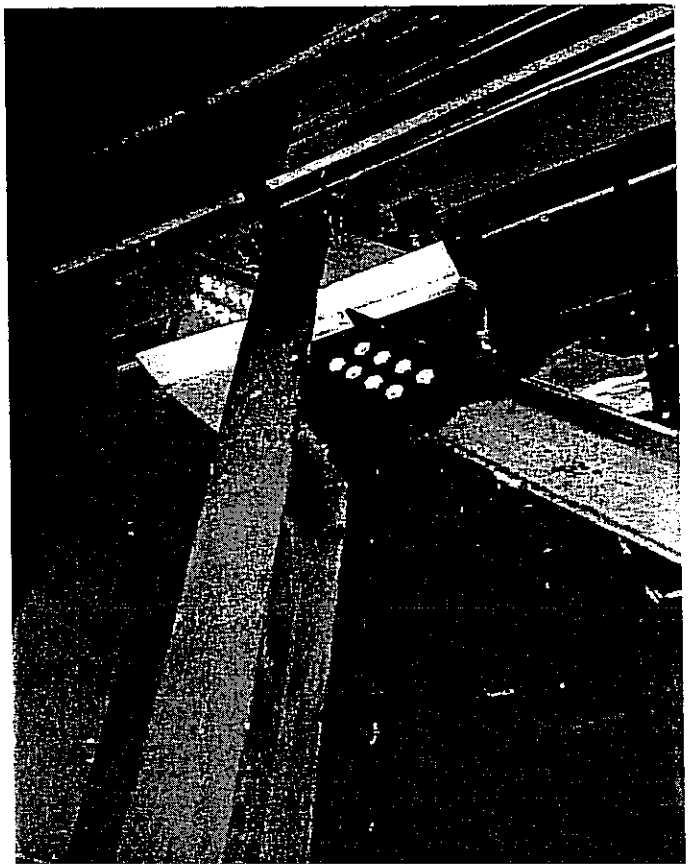


Photo 18. Typical connection of diagonal bracing to column top.



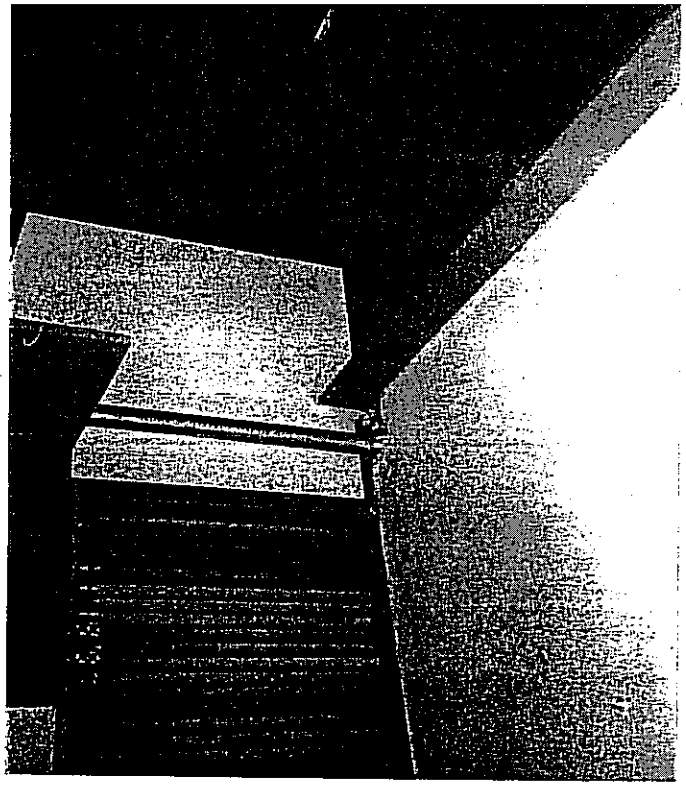
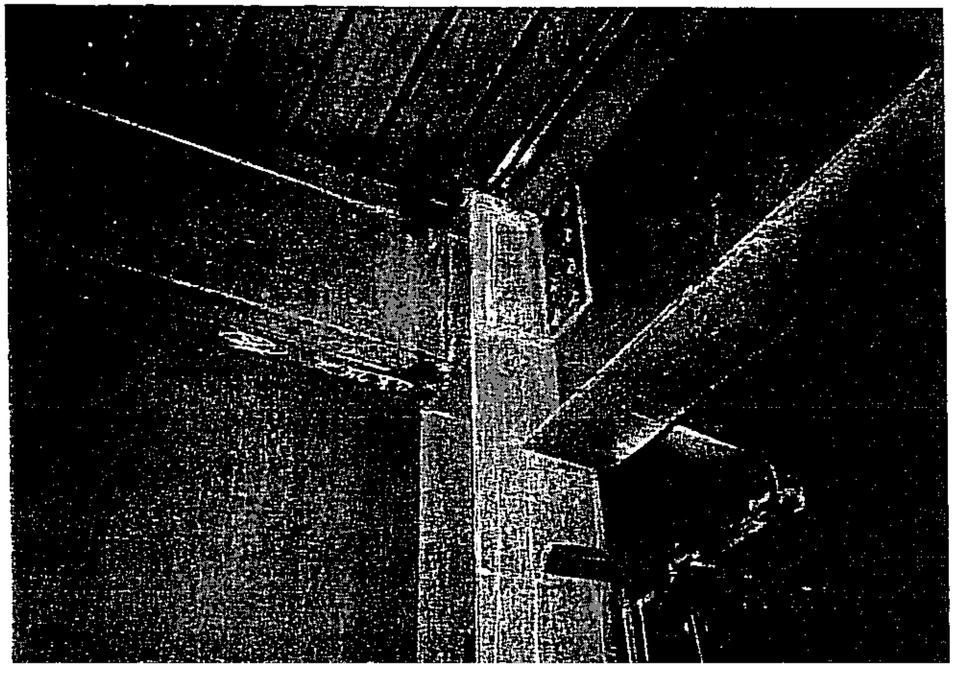


Photo 20. Typical connection of floor beams to column top. Notice the connection is not designed as a moment resistant connection. The steel deck floor is tack welded to the floor joist.



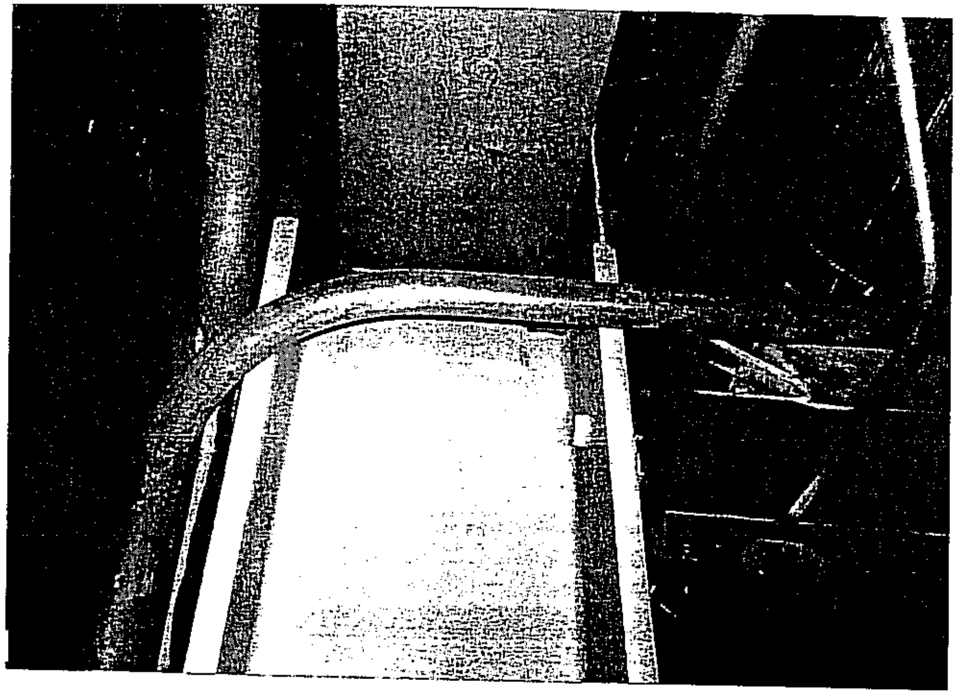
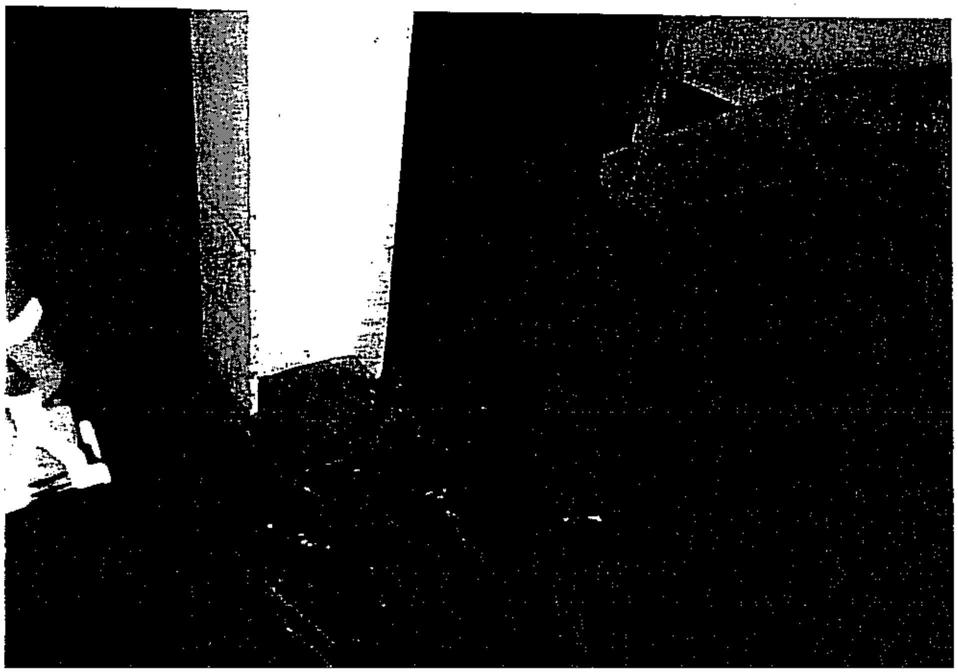
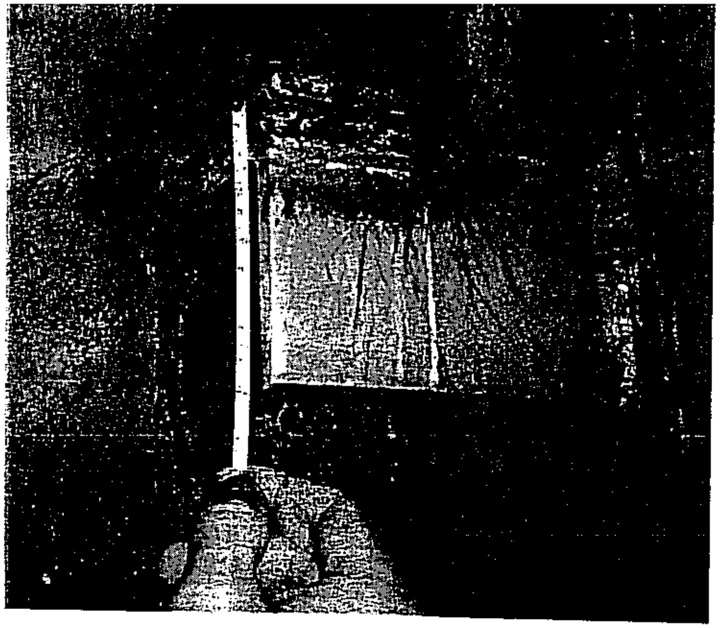
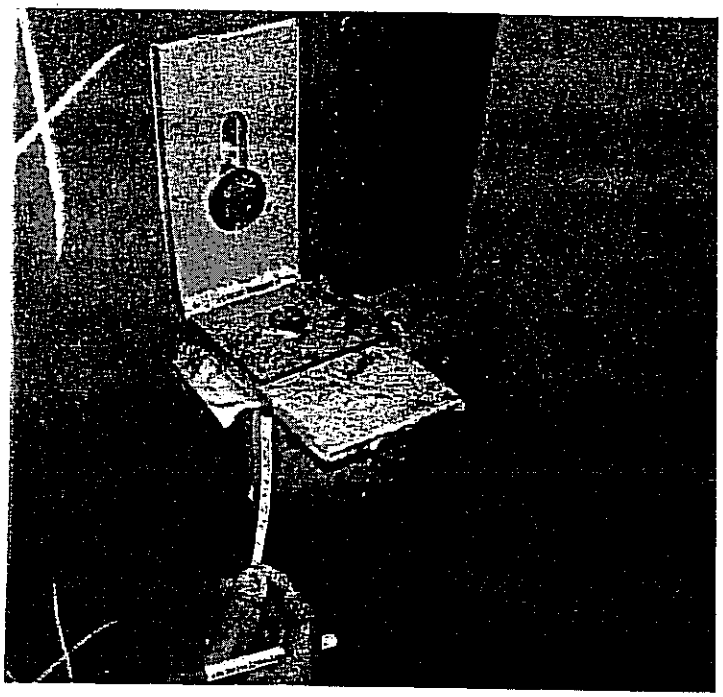
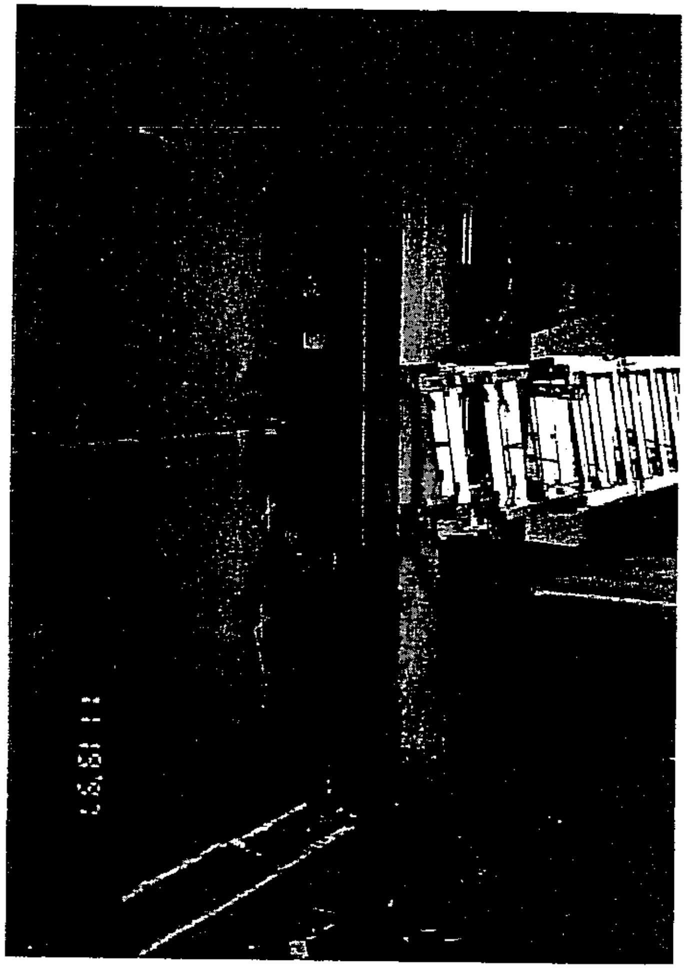


Photo 22. Another view of the connection of floor beams to column top. Again, notice the connection is not designed as a moment resistant connection.









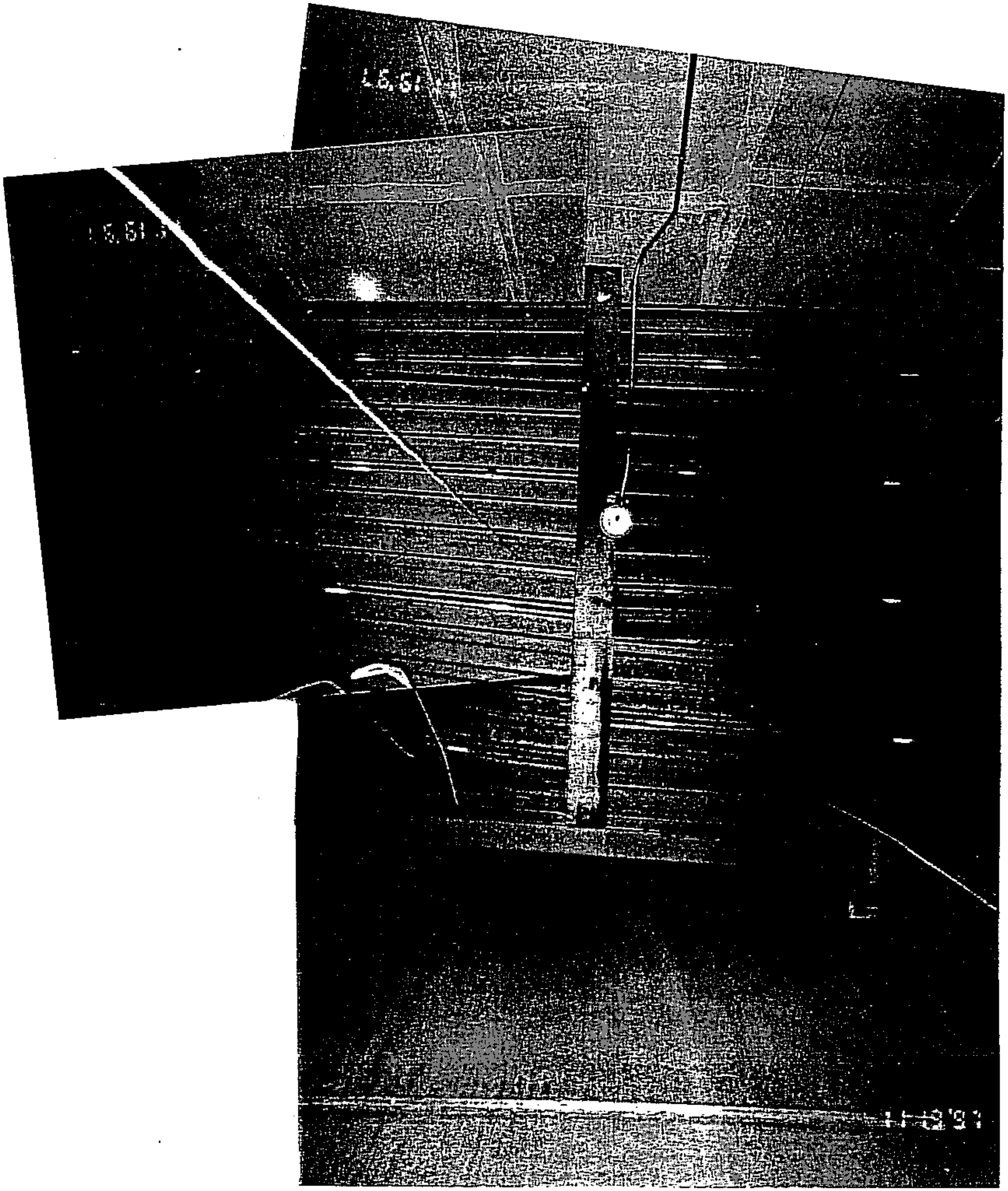


Photo 25   Underneath view of the typical steel deck floor. The floors consist of a concrete slab over a

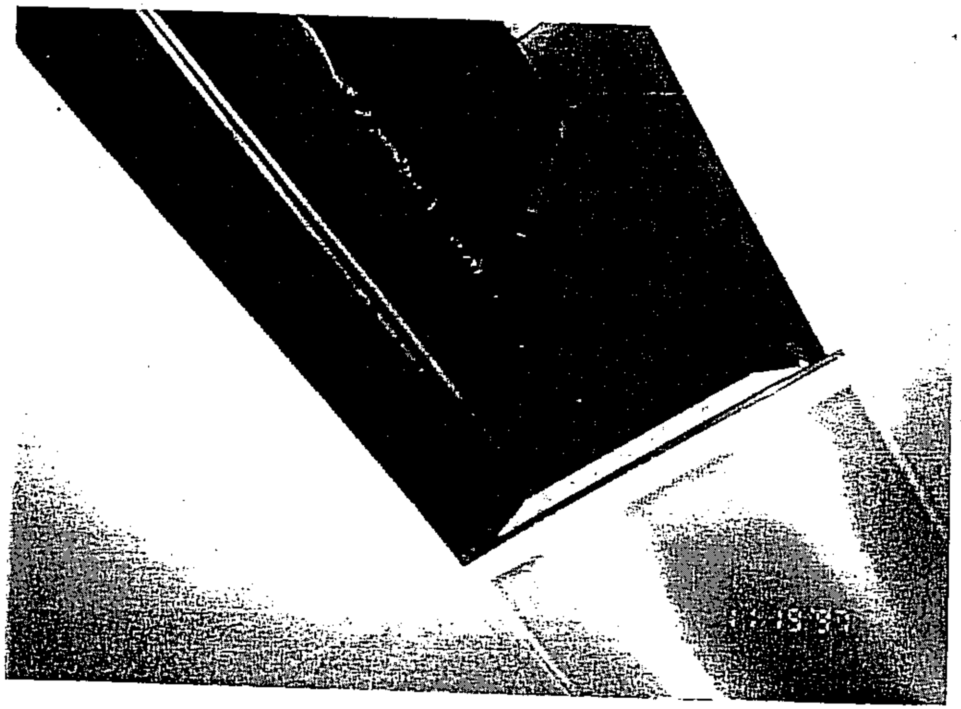
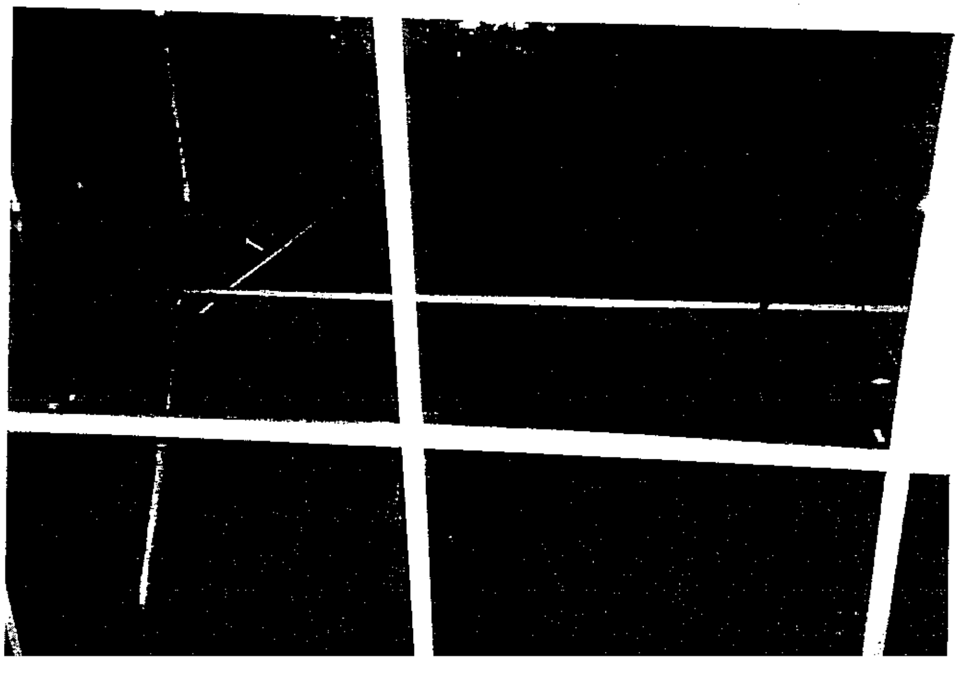


Photo 26. View of the ceiling, mechanical ducts and light fixture supports.



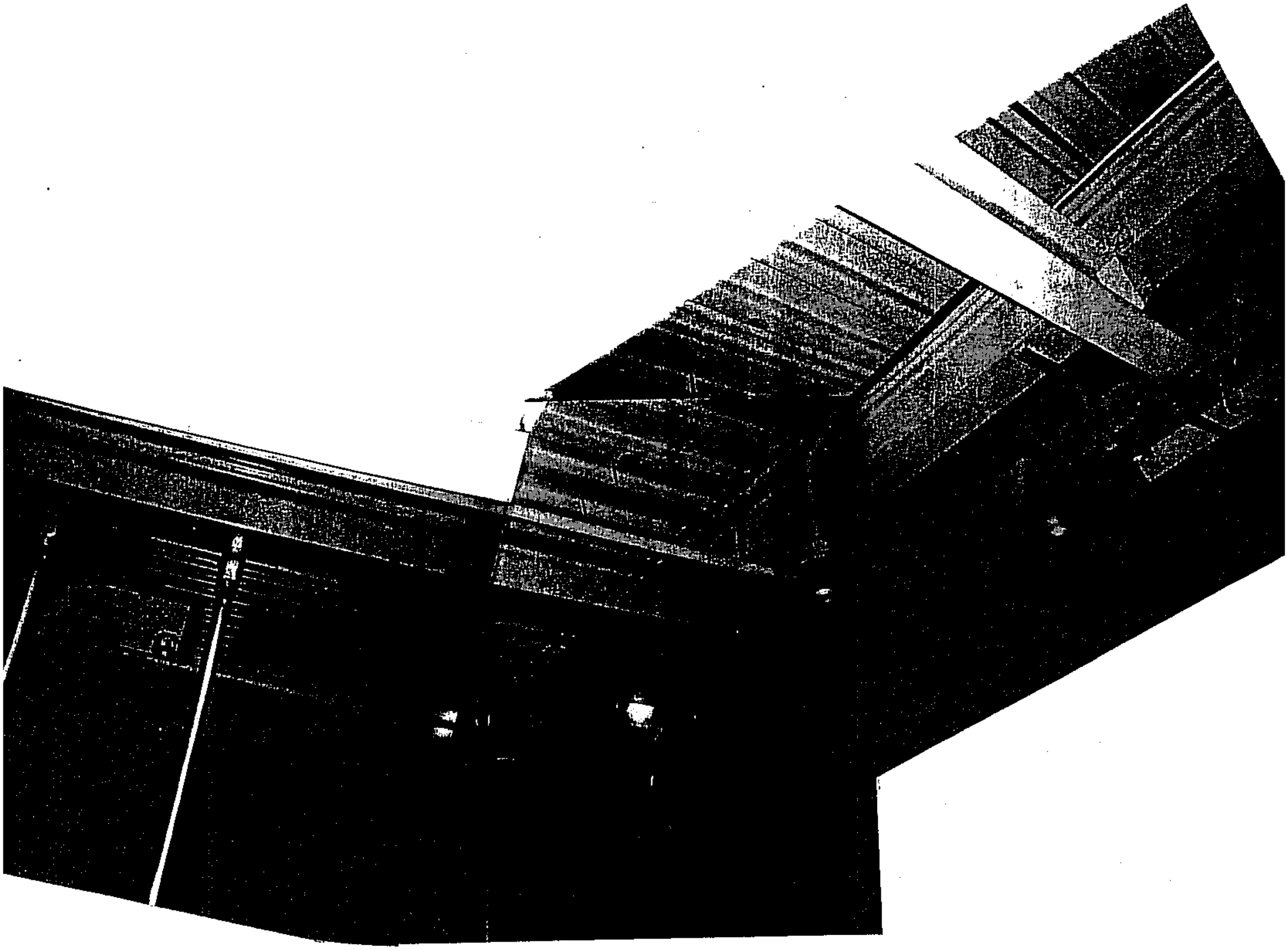


Photo 28. Underneath view of the steel deck roof. The roof consists of a built up roofing over a steel deck. The deck is tack welded to the roof joists. The roof offers limited diaphragm shear resistance to the building structure because of the fact that the tack welds do not qualify as a proper shear connection between the roof deck and the joists.

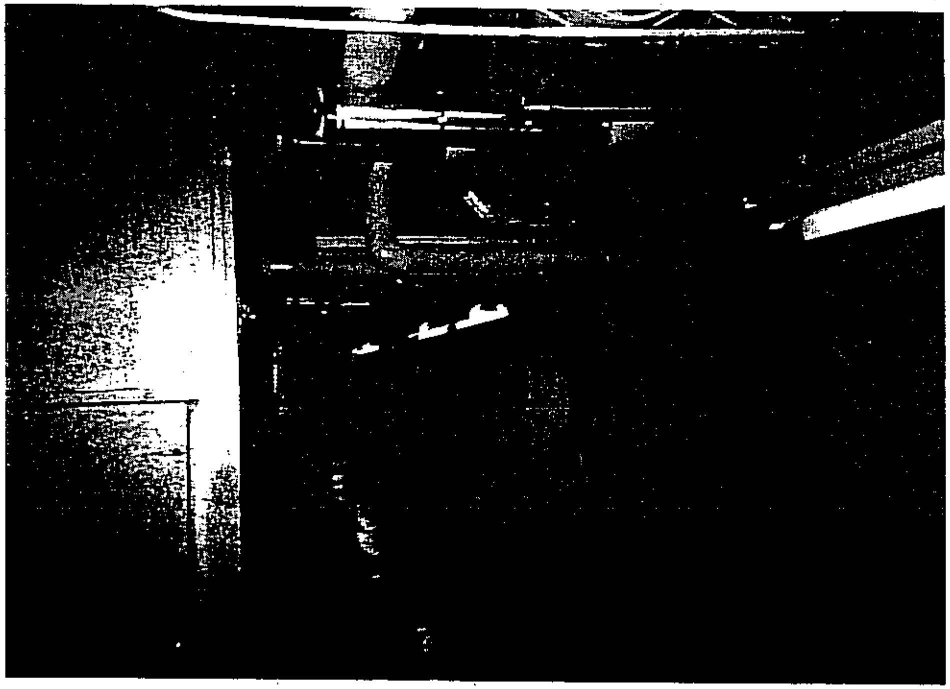
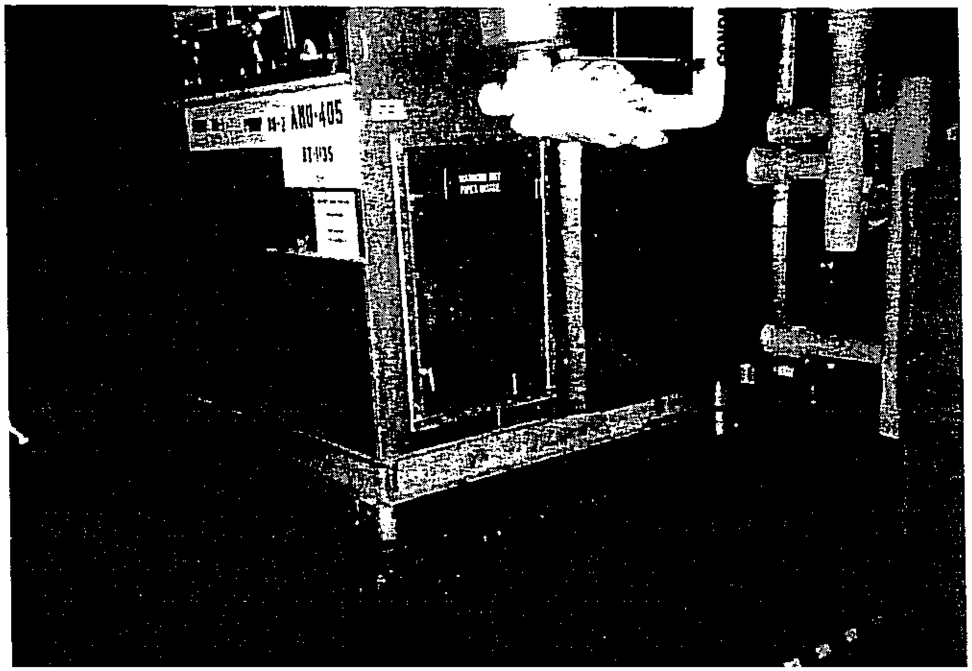


Photo 29. View of the Mezzanine floor. This floor hosts HVAC equipment as well as other electrical and mechanical equipment.



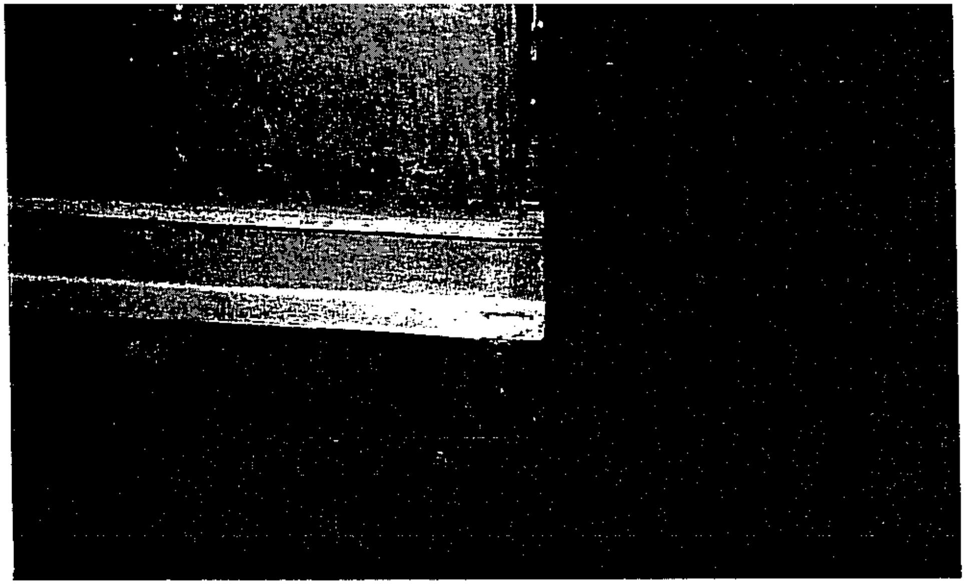
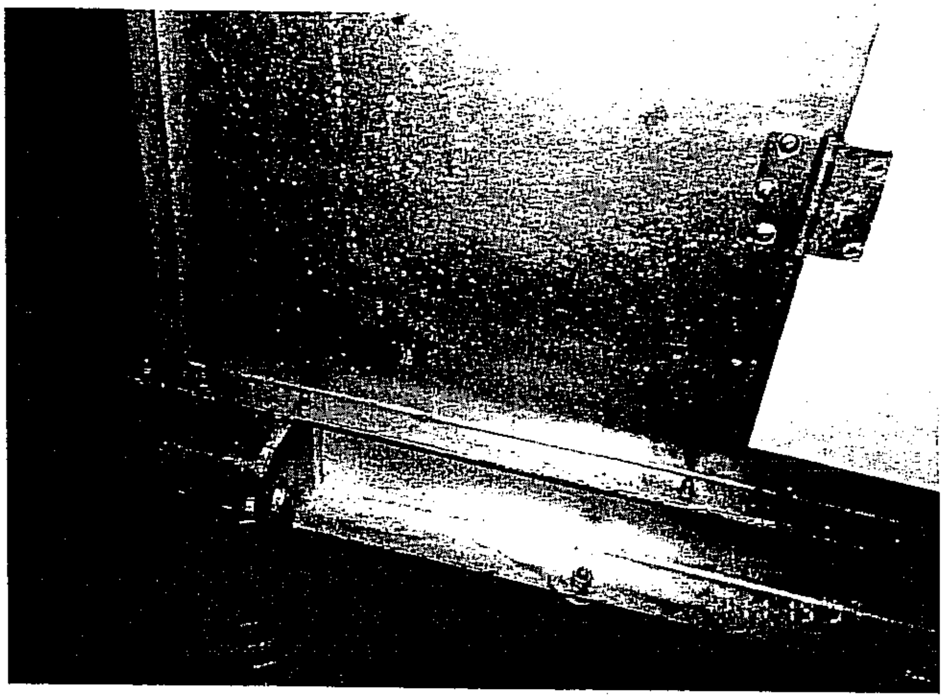


Photo 31. View of different type of anchorage for the mechanical equipment. This one is a rigid anchor type with no allowance for vibration. It connects the equipment rigidly to the foundation.



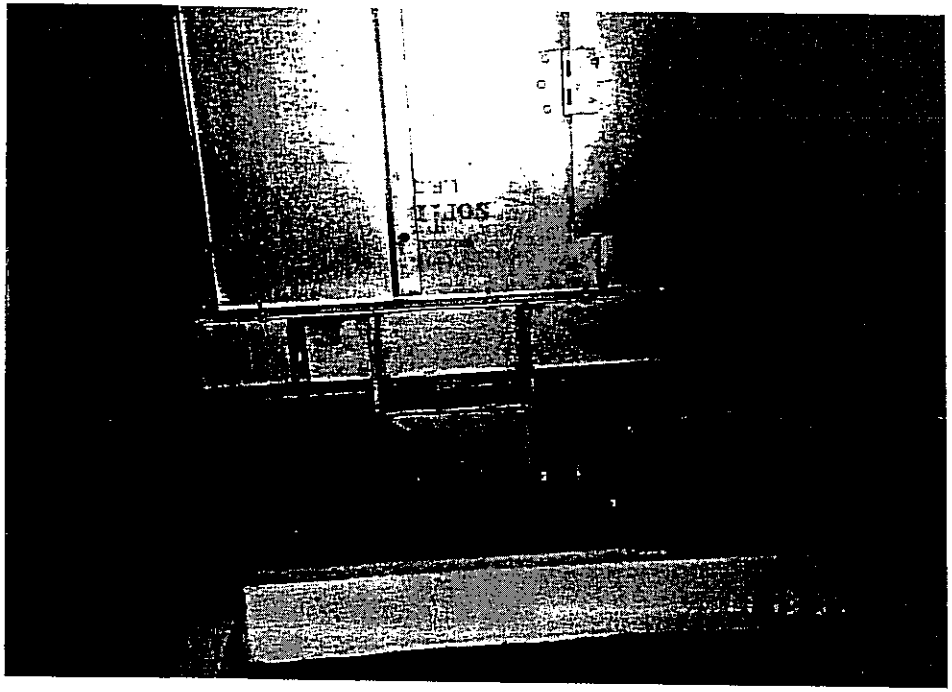


Photo 33. This other type of anchorage for the mechanical equipment provides vibration isolation and allows lateral restraint with a dampering effect. This anchor type is recommended to limit horizontal and vertical motion while absorbing stresses to minimize damage to equipment, piping connections or foundation.

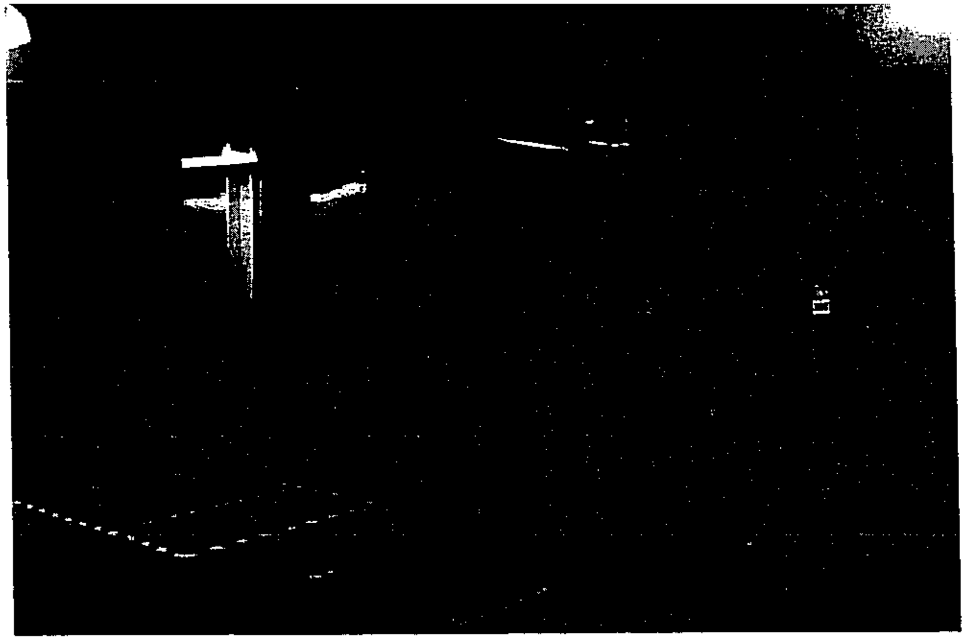
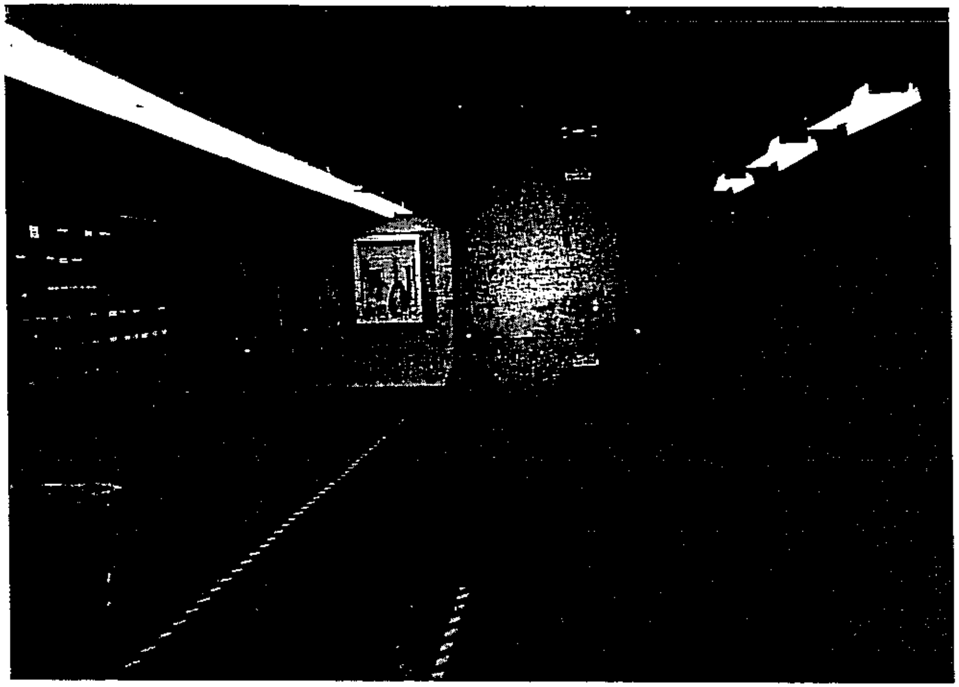


Photo 34. Electrical and mechanical equipment in the Mezzanine floor with no visible anchors. It is unknown whether the equipment is anchored and the anchors are concealed within the metal cabinetry.



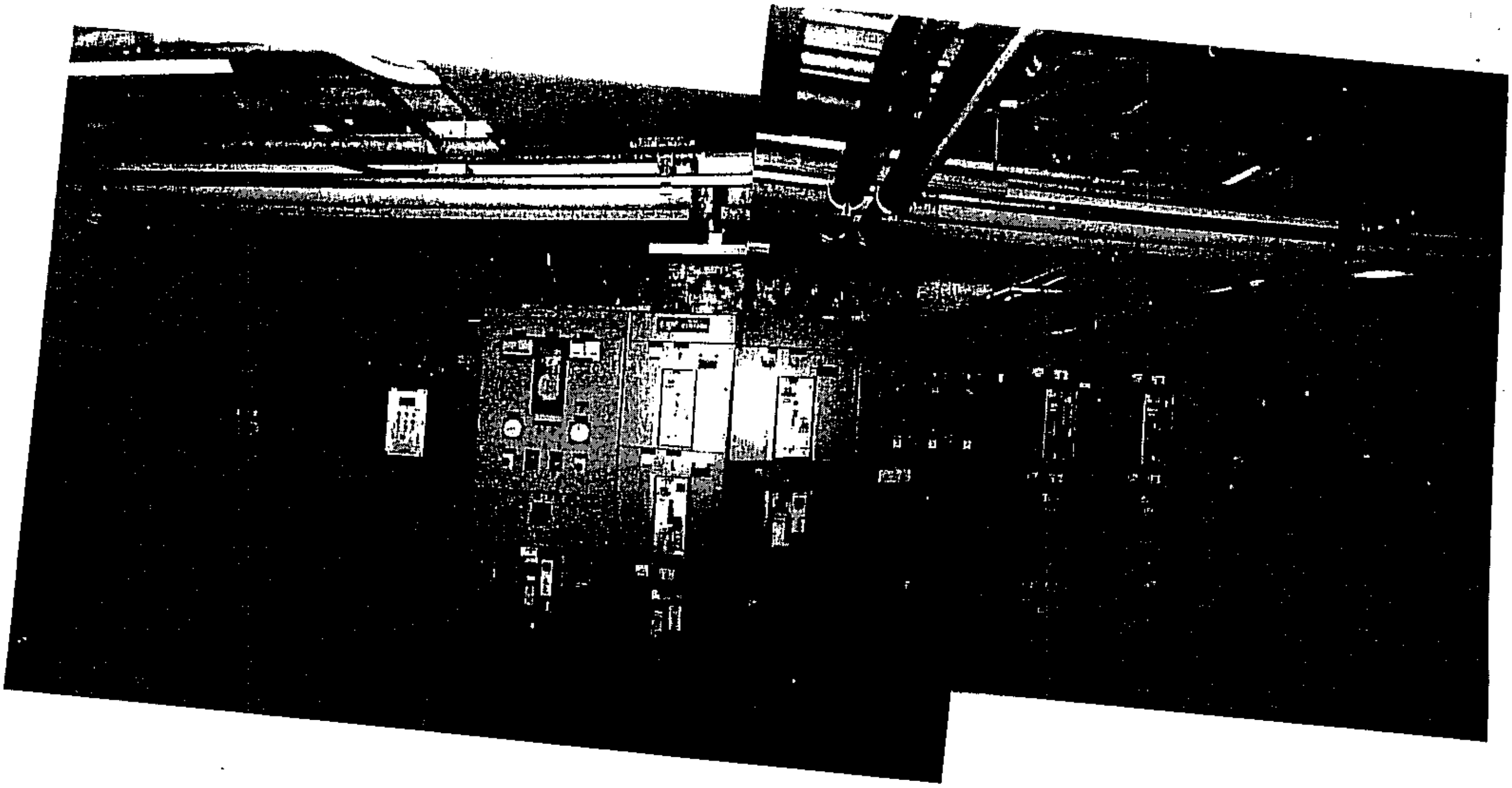
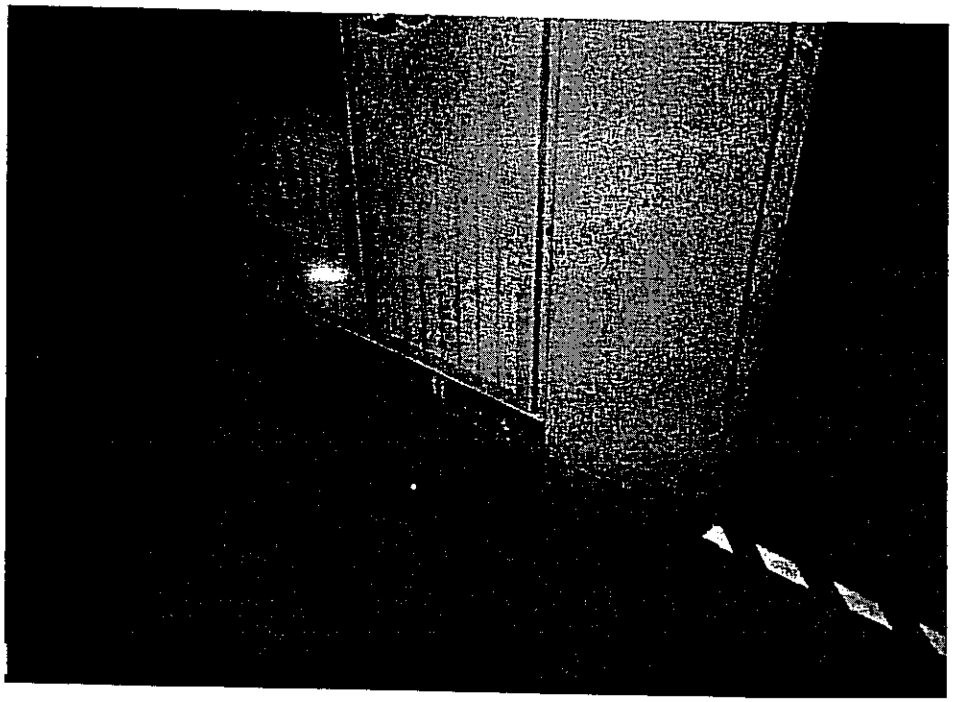


Photo 36. Electrical and mechanical equipment in the Mezzanine floor. It is unknown whether the equipment is anchored and the anchors are concealed within the metal cabinetry. The piping and conduits above are properly attached.

Photo 37. More equipment and furniture in the Mezzanine floor with rigid anchors directly attached to the floor. There is no pad under the equipment. The floor shows minor cracking as signs of distress caused by the anchors.



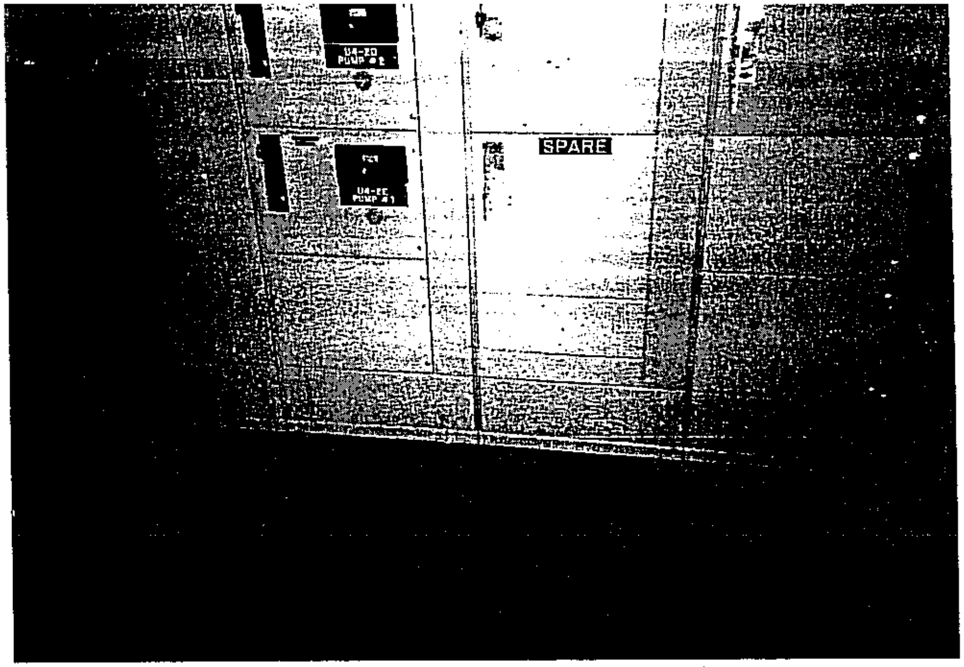
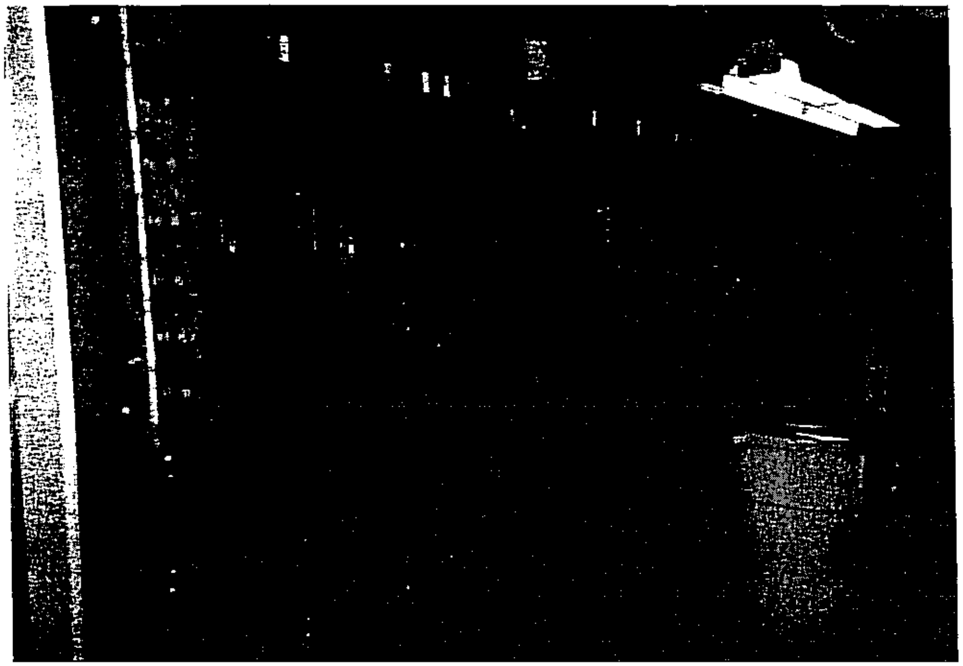


Photo 39. More equipment in the Mezzanine floor. It is unknown whether the equipment is anchored and the anchors are concealed within the metal cabinetry.



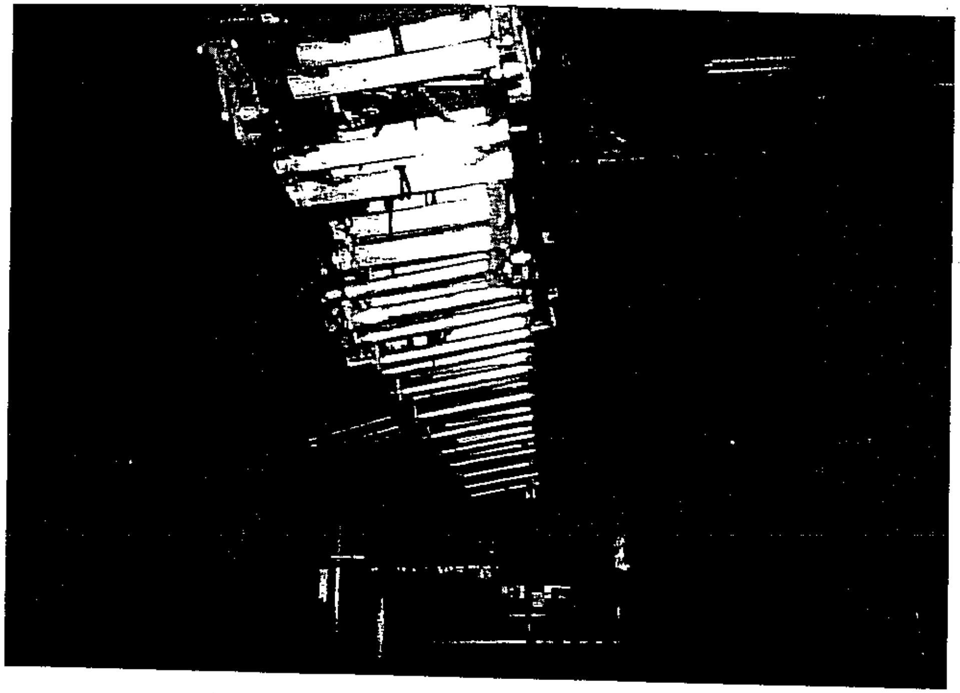
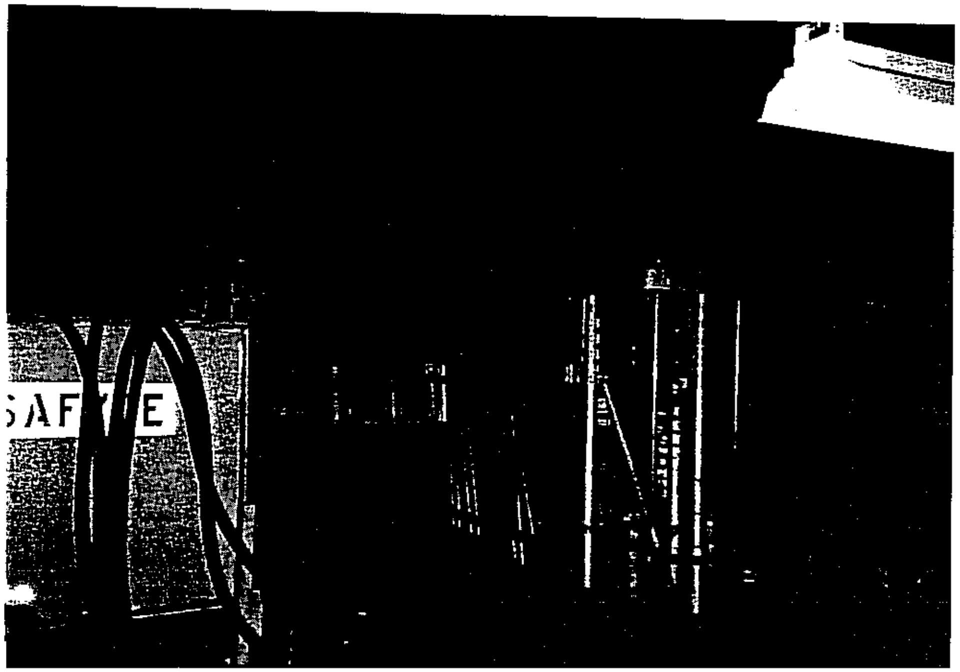


Photo 41. Ladders are hung from ceiling along pathways in the Mezzanine floor. They should be relocated against walls, away from pathways or escape exists, and properly secured.



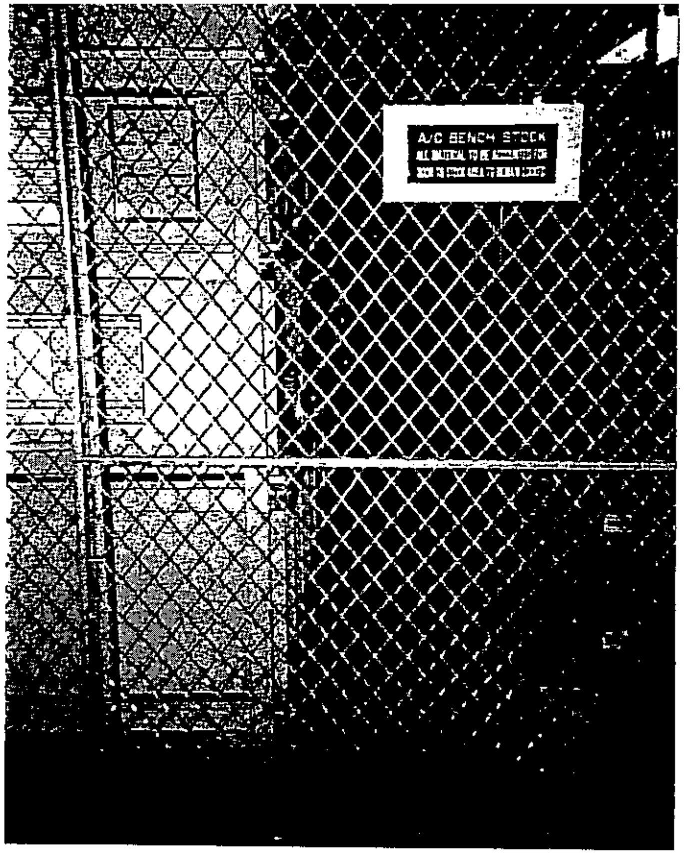
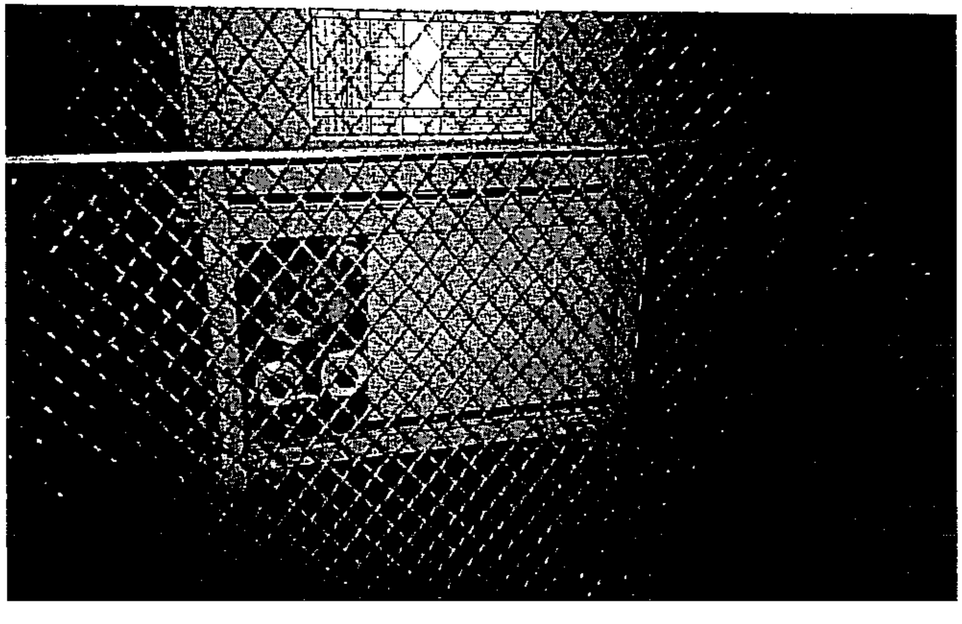


Photo 43. Designated equipment storage areas are fenced with chain-link fences properly anchored at the bottom. Anchoring of the top is recommended.



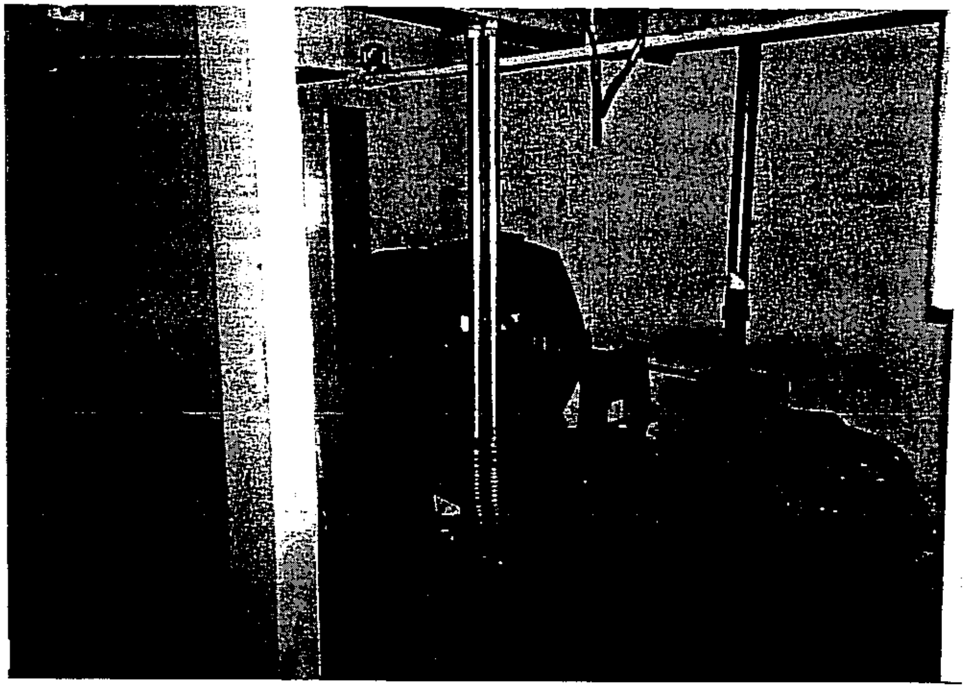
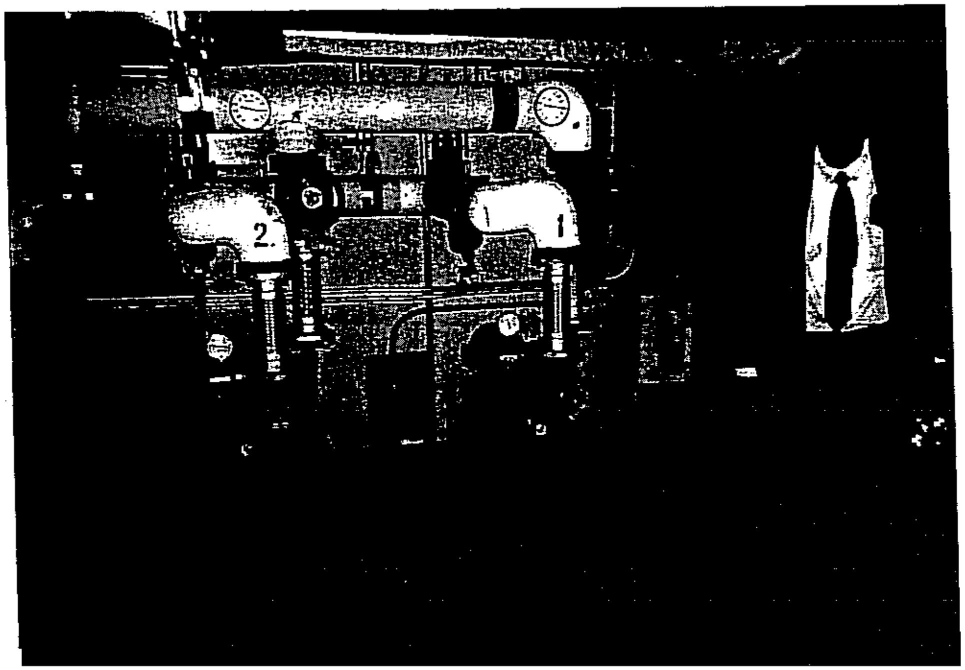


Photo 45. Elevator pumps and motors at elevator equipment room in the fourth floor are properly anchored.



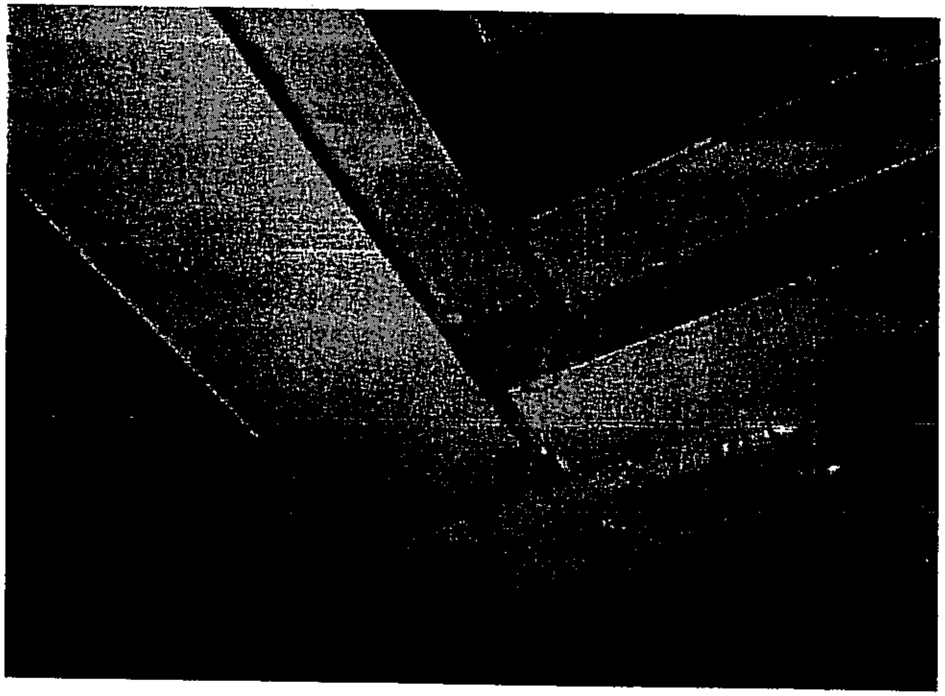
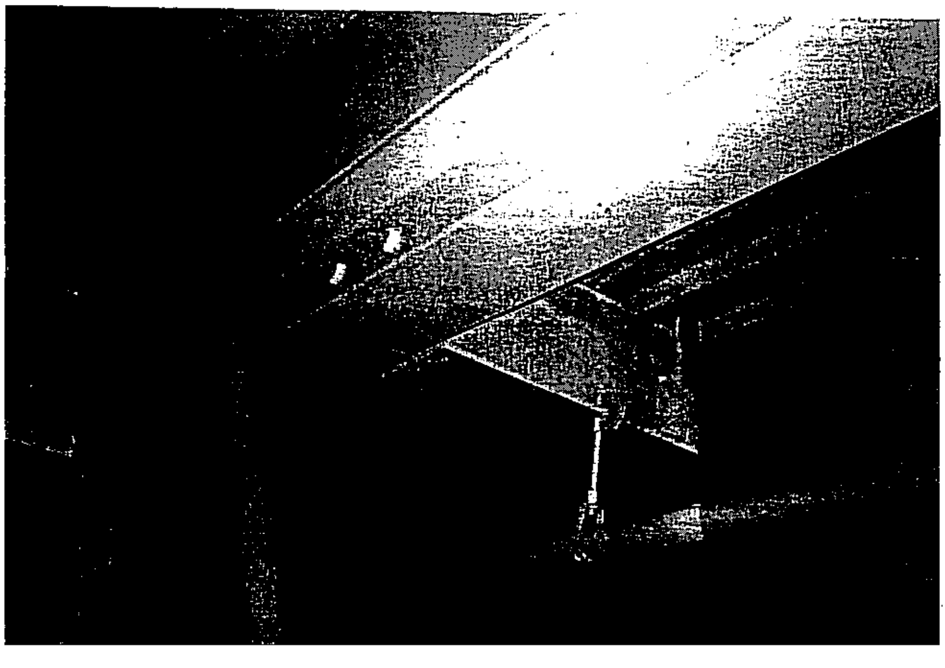


Photo 47. Some pipe crossing through walls do not have flexible joints or sleeves with tolerance for movement. Fire blocking is appropriate but tolerance for movement should be included.



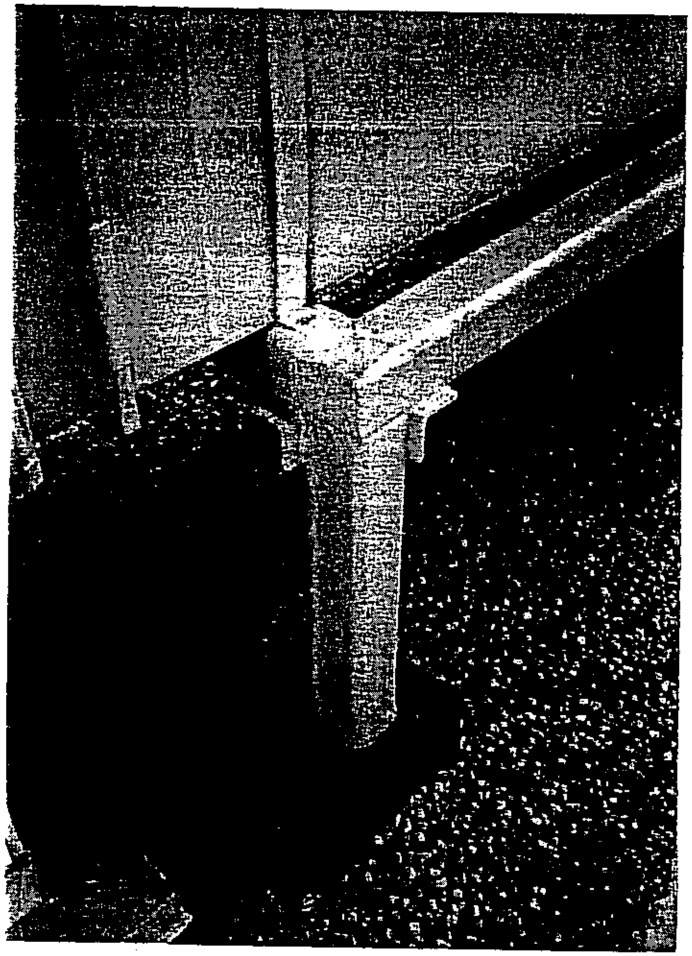


Photo 49. Some equipment supports at the roof level are rigidly anchored to the roof structure. Lateral bracing in both directions is recommended for legs supporting equipment over 18 inches above floor level.

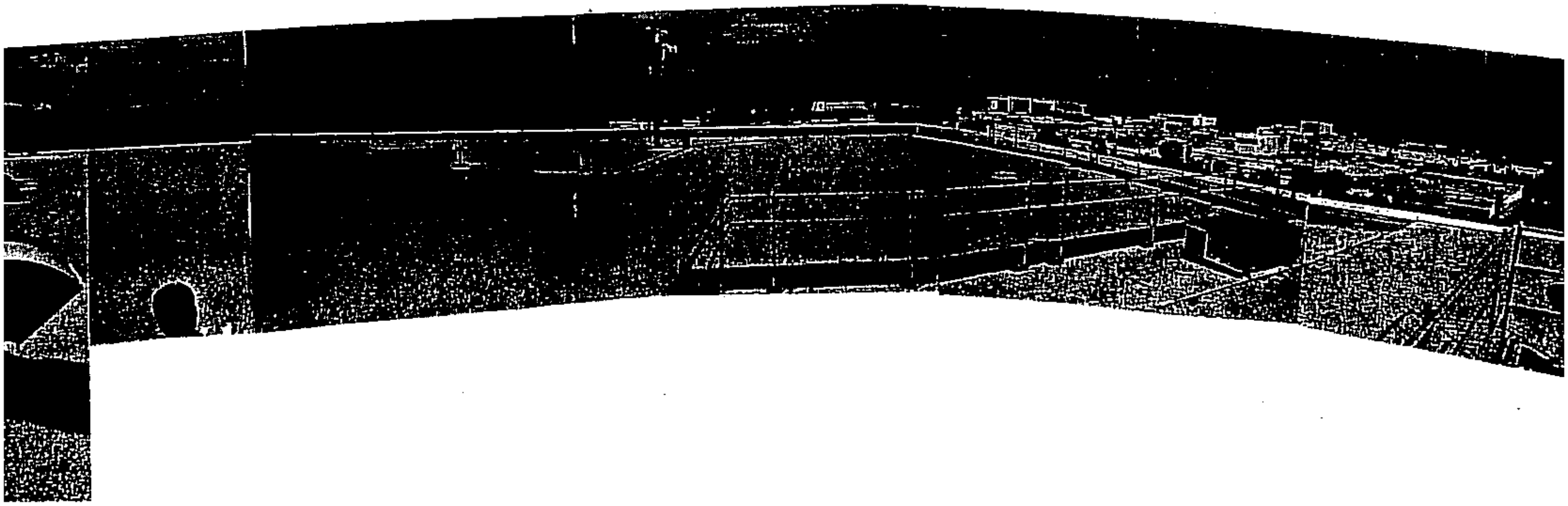


Photo 50. Miscellaneous equipment and antennas on the roof need to be secured more properly. Some redundant supports or anchorage are recommended.

DATA SUMMARY SHEET

BUILDING DATA

Year built: 1967 Year(s) remodelled: ______
Date of Evaluation: NOV-97
Area, (sq. ft.) 170,000 Length 258' Width 143' Photo Roll No. 1

CONSTRUCTION DATA

Roofframing: BUILT-UP ROOFING OVER STEEL DECK, OVER STL JOISTS.
Intermediate floor framing: CONC LAYER OVER STEEL DECK
Ground floor: CONC SLAB Basement: N/A
Exterior walls: PRECAST PANELS Openings: DOORS @ GROUND LEVEL
Columns: STEEL FRAME Foundations: CONC. PILES
General condition of structure: GOOD CONDITION
Evidence of settling: NONE

LATERAL FORCE RESISTING SYSTEM

	Transverse	Longitudinal
Model building type:	4 (Steel Braced Frame)	4 (Steel Braced Frame)
Building period, T:		

Unreduced base shear,

$V = [(0.80A_v \times S)/(R \times T^{2/3})] \times (W)$ or $V = [2.12Aa/R] \times W$ = 0.17

Response Modification Coefficient, R: 5 5

EVALUATION DATA

A_a = 0.4 A_v = 0.4

Site soil profile type: S-2 Site soil coefficient, S = 1.2

REMARKS

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigations, refer to the handbook section indicated in parentheses at the end of the statement.

CONDITION OF FOUNDATIONS

T F FOUNDATION PERFORMANCE: The structure does not show evidence of excessive foundation movement such as settlement or heave that would affect its integrity or strength. (Sec. 9.1.1)

T F DETERIORATION: There is no evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown, or other reasons in a manner that would affect the integrity or strength of the structure. (Sec. 9.1.2)

CAPACITY OF FOUNDATIONS

T F OVERTURNING: The ratio of the effective horizontal dimension, at the foundation level of the seismic resisting system, to the building height (base/height) exceeds $1.4A_v$. (Sec. 9.2.1)

T F TIES BETWEEN FOUNDATION ELEMENTS: Foundation ties adequate for seismic forces exist where footings, piles, and piers are not restrained by beams, slabs, or competent soils or rock. (Sec. 9.2.2)

T F LATERAL FORCE ON DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. (Sec. 9.2.3)

N/A T F POLE BUILDINGS: Pole foundations have adequate embedment. (Sec. 9.2.4)

N/A T F SLOPING SITES: The grade difference from one side of the building to another does not exceed one-half story. (Sec. 9.2.5)

GEOLOGIC SITE HAZARDS

T F LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 feet under the building. (Sec. 9.3.1) LIQUEFACTION POTENTIAL WITHIN 50' BELOW BUILDING EXISTS BUT FOUNDATION IS BASED ON A DEEP-PILE SYSTEM.

T F SLOPE FAILURE: The building site is sufficiently remote from potential earthquake

Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses after the headings.

PARTITIONS (Sec. 10.5.1)

T F In areas of high seismicity (A_v greater than or equal to 0.2), there are no unbraced unreinforced masonry or hollow clay tile partitions.

T F Partitions and fixed glass are detailed to accommodate the expected interstory drift.

T F Partitions at structural separations have seismic or control joints.

T F The tops of partitions that only extend to the ceiling line have lateral bracing.

CEILING SYSTEMS (Sec. 10.5.2)

T F Suspended ceilings and any ceiling-supported lighting or mechanical fixtures are adequately braced. HANGING FROM WIRES ONLY, NO LATERAL OR COMPRESSION MEMBERS.

T F Ceilings are not suspended plaster or gypsum board. (IN A FEW AREAS ONLY)

T F Lay-in tiles are not used for ceiling panels. – CEILINGS ARE MADE OF LAY-IN TILES

T F The edges of ceilings are separated from structural walls.

T F The ceiling system does not extend continuously across any seismic joints. – DIFFICULT TO VERIFY AT EVERY PLACE.

T F The ceiling system is not required to laterally support the top of gypsum board, masonry, or hollow clay tile partitions.

LIGHT FIXTURES (Sec. 10.5.3)

T F All light fixtures are supported and braced independently of the ceiling suspension system. – NOT TRUE AS SEEN ON 3RD FLOOR

T F Multiple length fluorescent fixtures have bracing or secondary support throughout their length. – NOT TRUE IN 3RD FLOOR & MEZZANINE.

T F The diffusers on fluorescent light fixtures are supplied with safety devices or some form of positive attachment. – [illegible]

(T) F Emergency lighting equipment and signs are anchored and/or braced to resist vertical and horizontal earthquake loads.

CLADDING, GLAZING, AND VENEER (Sec. 10.5.4)

(T) F All exterior cladding and veneer courses are properly anchored to the exterior wall framing for in-plane and out-of-plane lateral forces.

T F Masonry veneer is connected to the back-up with corrosion-resistant ties; in areas of high seismicity (A_v greater than or equal to 0.2), tie spacing is at 24 inches on center maximum with at least one tie for every 2-2/3 square feet. ← NO MASONRY VENEER EXISTS!

T F For moment frame buildings of steel or concrete, panels are isolated from the structural frame to absorb predicted interstory drift without collapse. N/A

(T) F Where multistory panels are attached at each floor level, the panels and connections can accommodate interstory drift.

(T) F Where bearing connections are required, there are at least two bearing connections for each wall panel.

(T) F Where inserts are used in concrete connections, the inserts are properly anchored to reinforcing steel.

(T) F There are at least four connections for each wall panel capable of resisting out-of-plane forces.

(T) F Welded connections appear to be capable of yielding in the base metal before fracturing the welds or inserts.

(T) F All eccentricities in connections are accounted for.

(T) F Connections appear to be installed properly.

(T) F No connection element is severely deteriorated or corroded. – SOME ARE SLIGHTLY RUSTED, RECOMMEND TO BE PAINTED.

(T) F There is no cracking in the panel materials indicative of substantial structural distress.

T (F) Glazing is isolated to accept predicted interstory drift without shattering. – NOT IN SOME PLACES

(T) F There is no substantial damage to exterior cladding due to water leakage.

(T) F There is no substantial damage to exterior cladding due to temperature movements.

Metal Stud Back-Up Systems, General (Sec. 10.5.4.1)

(T) F Additional steel studs frame window and door openings.

Masonry Veneer with Stud Back-Up (Sec. 10.5.4.2) — N/A
THERE IS NO MASONRY VENEER.

T F Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level.

T F Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing.

T F Masonry veneer is connected to the back-up with corrosion-resistant ties; in areas of high seismicity (A_v greater than or equal to 0.2), tie spacing is at 24 inches on center maximum and with at least one tie for every 2-2/3 square feet.

T F Weep holes are present and base flashing is installed.

T F For veneer with anchorage to back-up that does not meet the requirements for anchorage identified above, the computed tensile stresses in the veneer do not exceed the allowable for unreinforced brick as defined by ACI 530.

T F Mortar joints in the masonry veneer are well filled, and material cannot be easily scraped out from the joints.

Masonry Veneer with Concrete Block Back-Up (Sec. 10.5.4.3) — N/A

T F Masonry veneer more than 30 feet above the ground is supported by shelf angles or other elements at each floor level.

T F Masonry veneer is adequately anchored to the back-up at locations of through-wall flashing.

T F Masonry veneer is connected to the back-up with corrosion-resistant ties; in areas of high seismicity (A_v greater than or equal to 0.2), tie spacing is at 24 inches on center maximum and with at least one tie for every 2-2/3 square feet.

T F In areas of high seismicity (A_v greater than or equal to 0.2), the concrete block back-up qualifies as reinforced masonry.

T F The concrete block back-up is positively anchored to the structural frame at 4 feet maximum intervals along the floors and roofs.

T F Mortar joints in brick and block wythes are well filled, and material cannot be easily scraped from the joints.

Thin Stone Veneer Panels (Sec. 10.5.4.4) — N/A

T F Stone anchorages are adequate for computed loads.

T F There are no visible cracks or weak veins in the stone.

T F There is no visible deterioration of screws or wood at panel attachment points.

PARAPETS, CORNICES, ORNAMENTATION, AND APPENDAGES (Sec. 10.5.5)

T F There are no laterally unsupported unreinforced masonry parapets or cornices above the highest anchorage level with height/thickness ratios greater than 1.5 (2.5 if A_v is less than 0.3).

T F Concrete parapets with height/thickness ratios greater than 1.5 (2.5 if A_v is less than 0.3) have vertical reinforcement.

T F Cornices, parapets, signs, and other appendages that extend above the highest anchorage level or cantilever from exterior wall faces and other exterior wall ornamentation are reinforced and well anchored to the structural system. – EXCEPT FOR SOME ANTENNAS AT ROOF. 3FT HIGH PARAPET WITHOUT DIAG. BRACING.

N/A

CHIMNEYS (Sec. 10.5.6) — THERE IS NO CHIMNEYS

T F No unreinforced masonry chimney extends above the roof surface more than twice the least dimension of the chimney.

T F Masonry chimneys are anchored to the floor and roof.

MEANS OF EGRESS (Sec. 10.5.7)

T F Walls around stairs, elevator enclosures, and corridors are not hollow clay tile or unreinforced masonry.

T F Stair enclosures do not contain any piping or equipment except as required for life safety. THERE ARE WATER LINES. SLAB SLIGHTLY CRACKED DUE TO DIAG. SHEAR.

T F Veneers, cornices, canopies, and other ornamentation above building exits are well anchored to the structural system.

T F Lay-in ceiling boards and tiles used in exits or corridors are secured with clips.

T F Canopies are anchored and braced to prevent collapse and blockage of building exits.

BUILDING CONTENTS AND FURNISHINGS (Sec. 10.5.8)

T F Tall, narrow (height/depth > 3) storage racks, bookcases, file cabinets, or similar heavy items are anchored to the floor slab or adjacent walls.

T F Tall file cabinets are anchored to the floor slab or an adjacent partition wall.

T F File cabinets arranged in groups are attached to one another to increase their stability.

T F Cabinet drawers have latches to keep them closed during shaking.

T F Computers and communications equipment are anchored to the floor slab and/or structural walls to resist overturning forces. — DIFFICULT TO VERIFY ANCHORAGES IN SOME PLACES.

T F Computer access floors are braced to resist lateral forces. — THERE ARE SOME LOOSE FLOOR TILES.

MECHANICAL AND ELECTRICAL EQUIPMENT (Sec. 10.5.9)

T F Equipment is adequately anchored to the structure or foundation. — IN MEZZANINE, THERE ARE HEAVY AIR HANDLING UNITS NOT PROPERLY ANCHORED

T F Equipment mounted on vibration isolators is equipped with restraints or snubbers to limit horizontal and vertical motion. — NOT ALWAYS. NO RESTRAINTS ON SOME EQUIPMENT

T F Life-safety evacuation mechanical and electrical equipment is properly mounted to continue operation after an earthquake. — SOME FIRE EXTINGUISHERS SITTING ON FLOORS AT STAIRCASES NEED TO BE STORED IN CABINETS OR ATTACHED TO WALLS.

T F No pieces of major mechanical equipment are suspended from the structure without seismic bracing. — SOME CONCERN FOR HEAVY PUMP AT ELEVATOR ROOM @ 4th FLOOR

T F All electrical equipment is positively attached to the structural system.

T F All equipment supported on access floor systems either is directly attached to the structure or is fastened to a laterally braced floor system. DIFFICULT TO VERIFY EVERYWHERE.

PIPING (Sec. 10.5.10)

T F For fire suppression piping (e.g., sprinkler system piping including standpipes), risers are anchored and braced with flexible couplings to allow for building drift and floor movement due to building configuration or seismic separation.

T F Gas and oil piping is anchored and braced. — ANCHORED BUT NOT BRACED LATERALLY.

T F Shutoff devices are provided at building utility interfaces to shut off the flow of gas, high temperature energy, etc., in the event of earthquake-induced failure.

T F No pipes cross seismic joints without a flexible connector.

T F No pipes are supported by other pipes.

T F No pipe sleeve wall opening has a diameter of less than about 2 inches larger than the pipe. SOME PIPING HAS NO SLEEVES.

T F There are no unrestrained one-side C-clamps that support major piping.

DUCTS (Sec. 10.5.11)

T F Stair pressurization and smoke control ducts are braced to resist horizontal and vertical

HAZARDOUS MATERIALS (Sec. 10.5.12)

T F Compressed gas cylinders are restrained against motion. — NEED BETTER RESTRAINT IN MEZZANINE ROOM.

T F Laboratory chemicals stored in breakable containers are restrained from falling by latched doors, shelf lips, wires, or other methods. N/A

T F Piping containing hazardous materials is provided with shut-off valves or other devices to prevent major spills or leaks.

ELEVATORS (Sec. 10.5.13)

T F All elements of the elevator support system are anchored and configured to resist lateral seismic forces.

T F With the elevator car and/or counterweight located in its most adverse position in relation to the guide rails and support brackets, the horizontal deflection will not exceed 1/2 inch between supports and horizontal deflections of the brackets will not exceed 1/4 inch.

T F Snag points created by rail brackets, fish plates, etc., are equipped with guards to prevent the snagging of relevant moving elements.

T F The clearance between the car and counterweight assembly and between the counterweight assembly and the hoistway enclosure or separator beam is not less than 2 inches. VERY DIFFICULT TO CHECK!

T F Cable retainer guards on sheaves and drums are installed to inhibit the displacement of cables.

T F A retainer plate is provided at the top and bottom of both car and counterweight.

T F The clearance between the faces of the rail and the retainer plate does not exceed 3/16 inch.

T F The maximum spacing of the brackets that tie the counterweight rail to the building structure does not exceed 16 feet.

T F An intermediate spreader bracket is provided for tie brackets spaced greater than 10 feet and two intermediate spreader brackets are provided for tie brackets spaced greater than 14 feet.

T F The elevator motor is restrained by the vibration isolator system. — NOT TRUE AS SEEN ON 4TH FLOOR ELEVATOR EQUIPMENT ROOM.

T F The elevator control panel is anchored at top and bottom.

These buildings are similar to Type 3 buildings except that the vertical components of the lateral-force-resisting system are braced frames rather than moment frames.

> Address the following evaluation statements, marking each either true (T) or false (F). Statements that are found to be true identify issues that are acceptable according to the criteria of this handbook; statements that are found to be false identify issues that need investigation. For guidance in the investigation, refer to the handbook section indicated in parentheses at the end of the statement.
>
> Be advised that the numerical indices preceded by an asterisk (*) in these statements are based on high seismicity (A_v = 0.4). Adjustments are reasonable for lower seismicity. The appropriate adjustment is not necessarily a direct ratio of seismicity.

BUILDING SYSTEMS

T F LOAD PATH: The structure contains a complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation (NOTE: Write a brief description of this linkage for each principal direction.) (Sec. 3.1) E W and N S perimeter frames have diagonal braces

T F REDUNDANCY: The structure will remain laterally stable after the failure of any single element. (Sec. 3.2)

T F WEAK STORY: Visual observation or a Quick Check indicates that there are no significant strength discontinuities in any of the vertical elements in the lateral-force-resisting system; the story strength at any story is not less than 80 percent of the strength of the story above. (Sec. 3.3.1)

T F SOFT STORY: Visual observation or a Quick Check indicates that there are no significant stiffness discontinuities in any of the vertical elements in the lateral-force-resisting system; the lateral stiffness of a story is not less than 70 percent of that in the story above or less than 80 percent of the average stiffness of the three stories above. (Sec. 3.3.2)

T F GEOMETRY: There are no significant geometrical irregularities; there are no setbacks (i.e., no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to the adjacent stories). (Sec. 3.3.3)

T F MASS: There are no significant mass irregularities; there is no change of effective mass of more than 50 percent from one story to the next, excluding light roofs. (Sec. 3.3.4)

T F VERTICAL DISCONTINUITY: All frames are continuous to the foundation. (Sec. 3.3.5) distance between the story center of rigidity and the story center of mass is greater than 20 percent of the width of the structure in either major plan dimension. (Sec. 3.3.6)

T F DETERIORATION OF STEEL: There is no significant visible rusting, corrosion, or other deterioration in any of the steel elements in the vertical or lateral-force-resisting systems. (Sec. 3.5.3)

BRACED FRAMES

T F STRESS CHECK: The building satisfies the Quick Check of the stress in the diagonal bracing. (Sec. 6.1.1)

T F STIFFNESS OF DIAGONALS: All diagonal elements required to carry compression have *Kl/r* ratios less than 120. (Sec. 6.1.2)

T F TENSION-ONLY BRACES: Tension-only braces are not used as the primary diagonal bracing elements in structures over two stories in height. (Sec. 6.1.3)

T F CHEVRON BRACING: The bracing system does not include chevron, V-, or K-braced bays. (Sec. 6.1.4)

T F CONCENTRIC JOINTS: All the diagonal braces frame into the beam-column joints concentrically. (Sec. 6.1.5)

T F CONNECTION STRENGTH: All the brace connections are able to develop the yield capacity of the diagonals. (Sec. 6.1.6)

T F COLUMN SPLICES: All column splice details of the braced frames can develop the column yield capacity. (Sec. 6.1.7)

DIAPHRAGMS

N/A T F PLAN IRREGULARITIES: There is significant tensile capacity at re-entrant corners or other locations of plan irregularities. (Sec. 7.1.1)

T F REINFORCING AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. (Sec. 7.1.3)

T F OPENINGS AT BRACED FRAMES: Diaphragm openings immediately adjacent to the braced frames extend less than 25 percent of the length of the bracing. (Sec. 7.1.5)

CONNECTIONS

T F TRANSFER TO STEEL FRAMES: The method used to transfer diaphragm shears to the steel frames is approved for use under lateral loads. (Sec. 8.3.2)

STRUCTURAL CALCULATIONS

original design Dec, [illegible] by
C F Braun & Co;

- UBC for seismic, no snow load
- AISC
- ACI
- air force manual 88-3
- Foundation Investigation: Harding Ass.
- SFF Design Criteria by Aerospace Corp.

Materials:

steel sections A 36

" Plates A 373

Bolts A 307 & A 325, threads in shear Plane

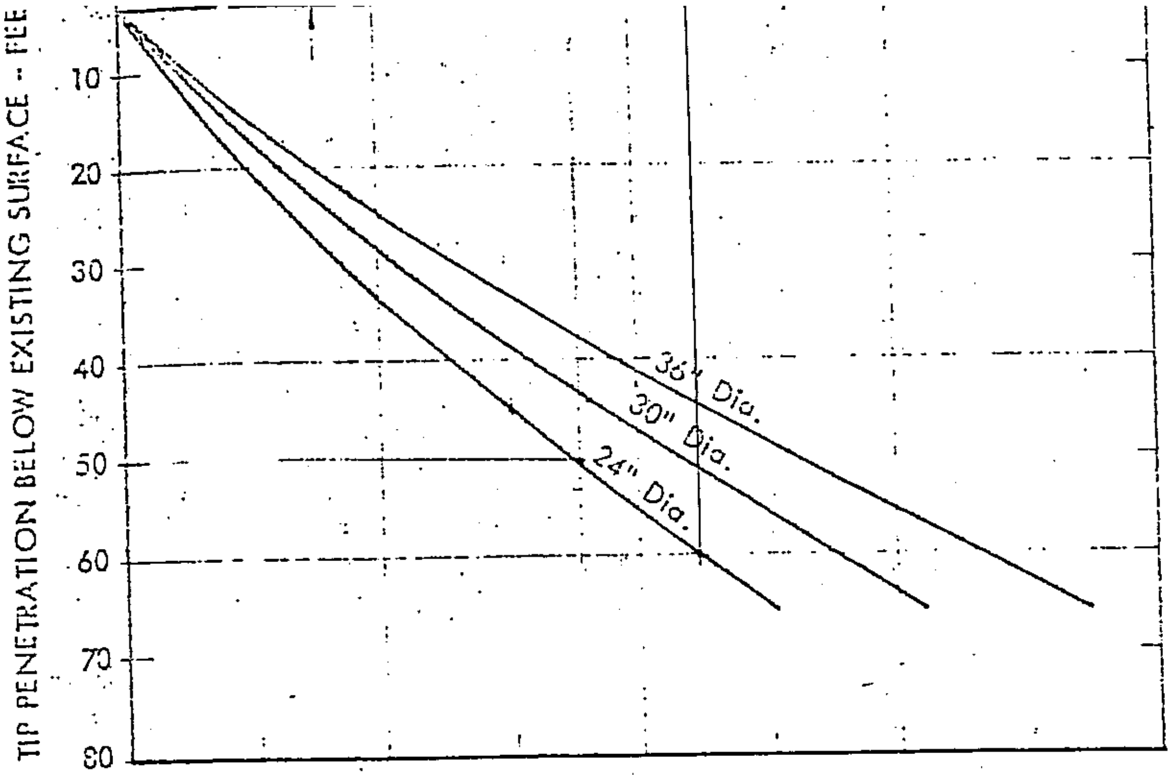
Concrete 3000 psi

Roofing	6 psf
Roof insulation	3 psf
Floors (4" composite)	44 psf
Equipment e/ partit	20 psf
fixed partitions	16 p.f.ht
computer floor	12 psf
cables	4.5 psf

Ceiling

ceiling	1.5
ducts	1.0
sprinkler	1.0
lights etc	-
	3.5 psf

Exterior wall panels	38.5 psf
2nd floor Mezz	20 psf.



DRILLED CAST -IN-PLACE CONCRETE PILES

NOTES:

1. The indicated capacities are for dead plus live loads and may be increased by one-half for total design loads including seismic forces.

2. The indicated capacities are based on the strength of the supporting soil; the structural capacity of the pile material may impose further limitations.

3. For uplift capacities use one-half of the indicated downward capacity (neglecting the one-half increase).

4. Piles installed in groups shall be spaced no closer than three diameters center to center.

5. The lengths shown are for average soil conditions as encountered in Boring 6 and in the test piles.

- Drilled cast in place 24" ø to 36" ø
45 ft to 60 ft long.
see capacity chart

- spread ftgs allowable bearing 3.0 Ksf
(not used)

Building Description:

The building is a braced steel frame structure as described in section 2 of the report.

The roof is metal decking.

- Roof 145 x 260 x 25 psf = 931
Wall Bracing
Elevator, Mech rooms = 77
Interior cols = 21
Ext steel of concrete = 558

1,587 K

- 4th Floor 37 228 x 98 psf = 3698
raised floor = 117
interior cols = 34
ext steel of p.c. walls = 948

4,747 K

- 3th Floor = 3698
raised floor = 117
interior columns = 69
ext. steel of p-c walls = 695

ext. columns = 62

new office area (37 228 - 15 795) 80 psf = 1,719

ext steel of p.c. walls = 619 K

3518 K

- 2nd Floor

37,228 sf × 98 psf = 3648

conc. raised flr 117

interior columns 84

ext steel of p.c. walls 841

4690 K

Total Building Weight 19,071 K

per section 2.4.3.2

$$T = 0.10 * N \qquad (N = \text{number of stories})$$

$$= 0.4 \text{ sec}$$

check using FEMA 178 equation 2-6G

$$T_a = \frac{0.05\, h_n}{\sqrt{L}}$$

$$= \frac{(.05)(100)}{\sqrt{143}}$$

$$T_a = 0.411 \text{ sec} \checkmark$$

use $T = 0.4$ sec

$V = 0.17\ W$

from page 5 $\quad W = 19{,}071^K$

To approximate the effect of torsion using a 5% increase for accidental torsion

$$V = 0.17 \times 1.05 \times 19{,}071$$

$$V = 3{,}404^K$$

Checking the braces in the 1st floor, 10 braces take the total base shear

∴ approximate shear per brace $= 340^K$

Brace capacity:

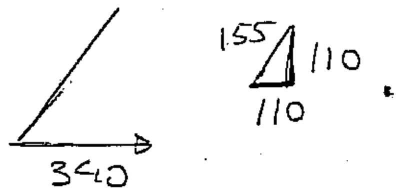
brace section $= 2(4 \times 4 \times 5/16)$ Typ angles

$A = 10 \text{ in}^2$

for A 36 steel

$C = 10 \times .67 \times 36$

$\underline{C = 241^K}$



Brace demand

$Q = 340^K \left(\frac{155}{110}\right)$

$\underline{Q = 479^K}$

Q/ 1 98

Pile capacity (neglect small contribution from the intake dead load)

$$\begin{Bmatrix} = 130\text{ Tons} = 260^K \\ = 90\text{ Tons} = 180^K \end{Bmatrix} \quad \begin{matrix} 36''\phi \\ 24''\phi \end{matrix}$$

increase by 50% for seismic load compression

$C = (1.5)(180)$ or $(1.5)(260)$

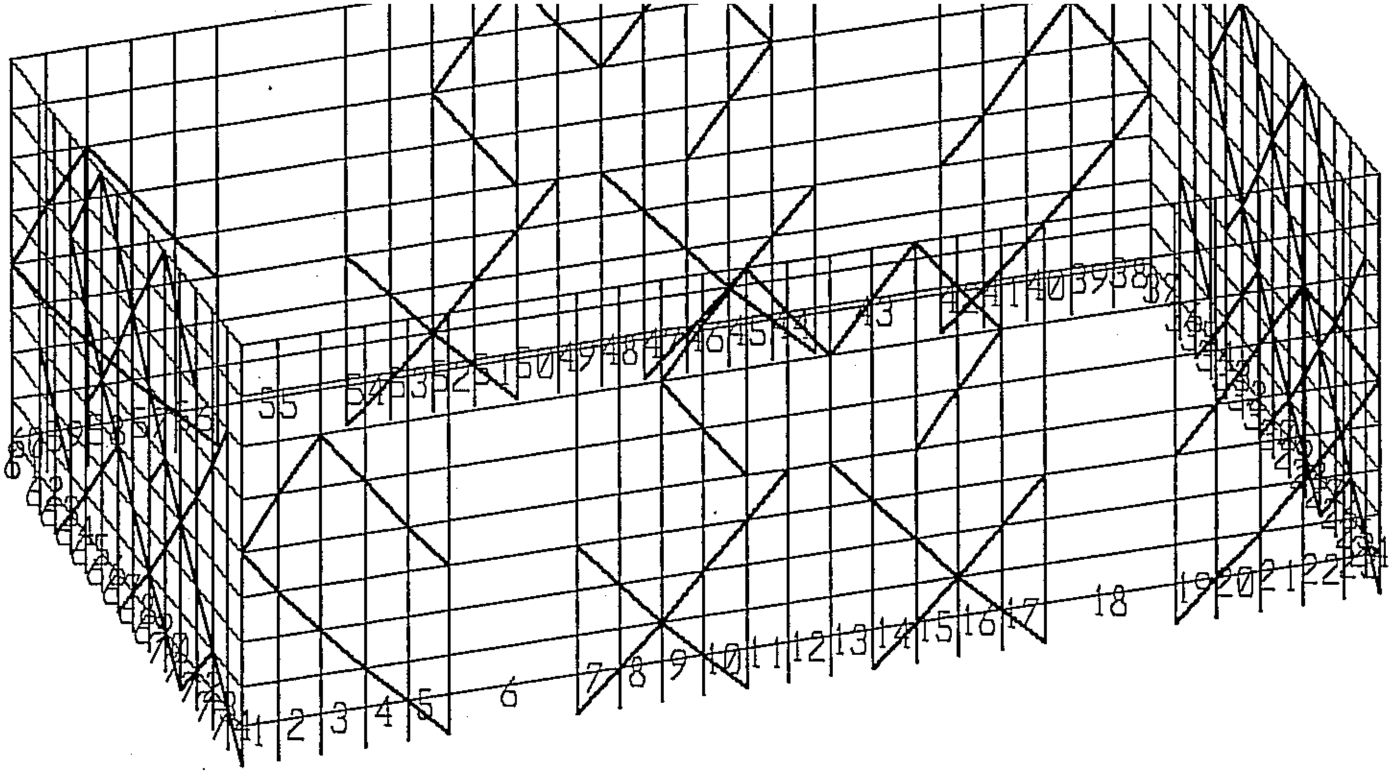
$C = 270^K$ for $24''\phi$ $\qquad$ $C = 390^K$ for $36''\phi$

$Q/C = 1.26$ in compression for $24''\phi$

$= 0.87$ " for $36''\phi$

in tension (uplift) neglect 50% increase and reduce by 50%

$Q/C = 3.77$ in uplift for $24''\phi$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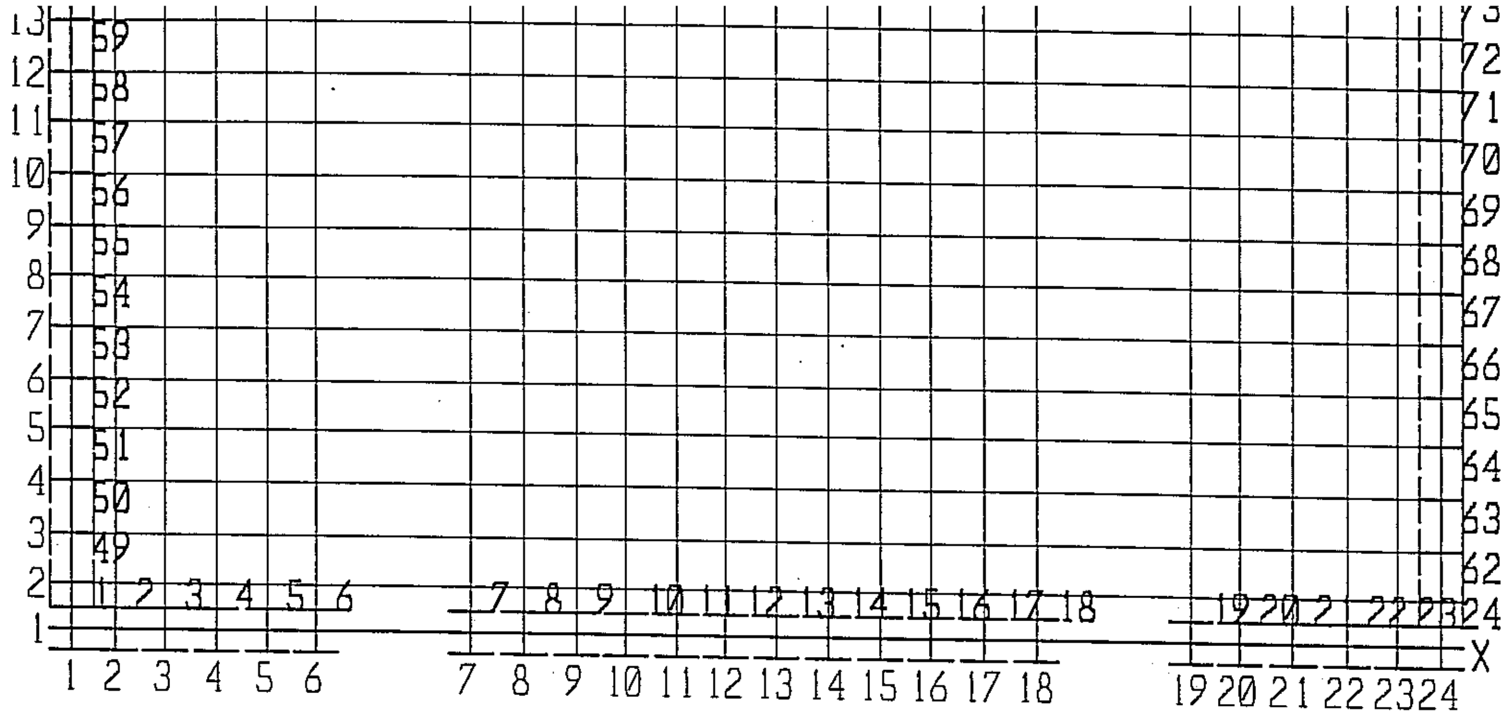
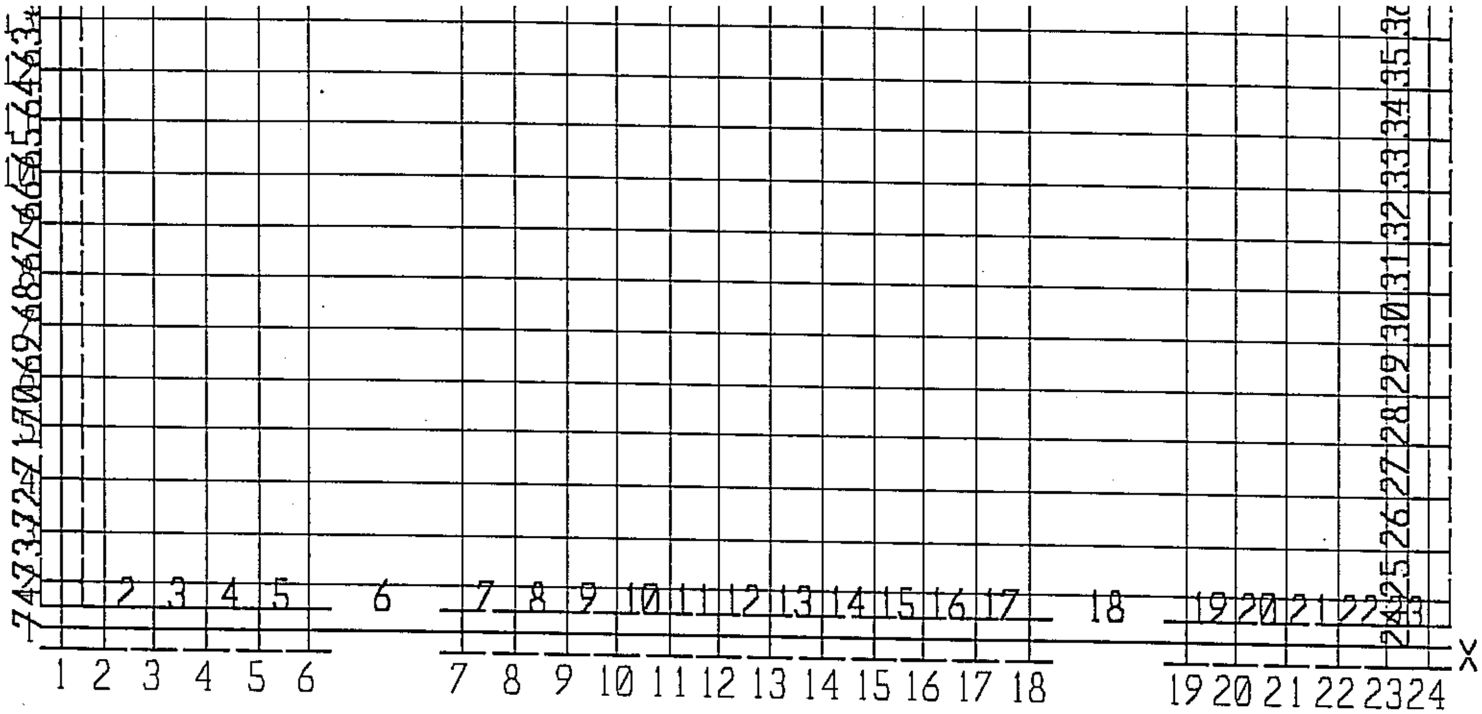
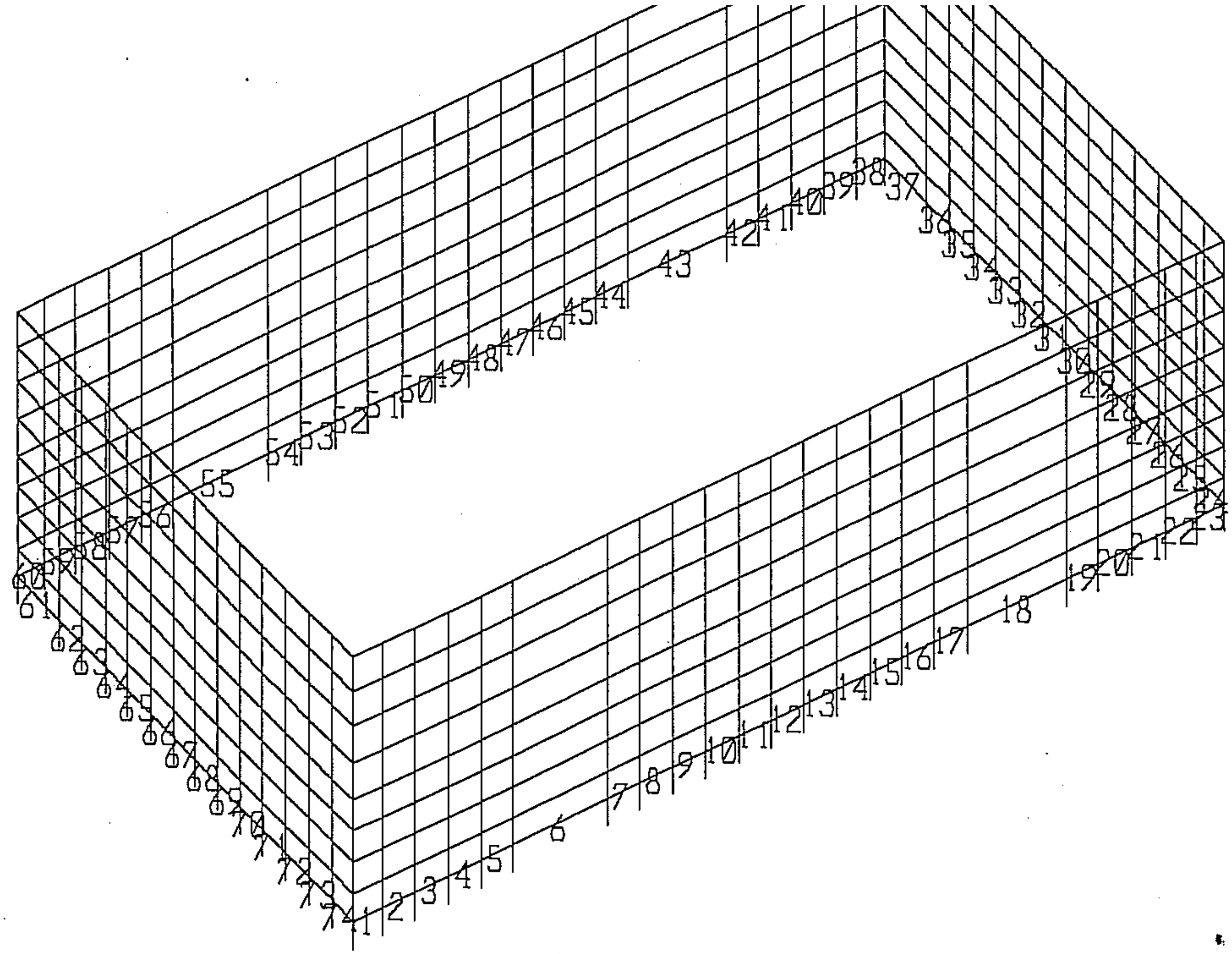
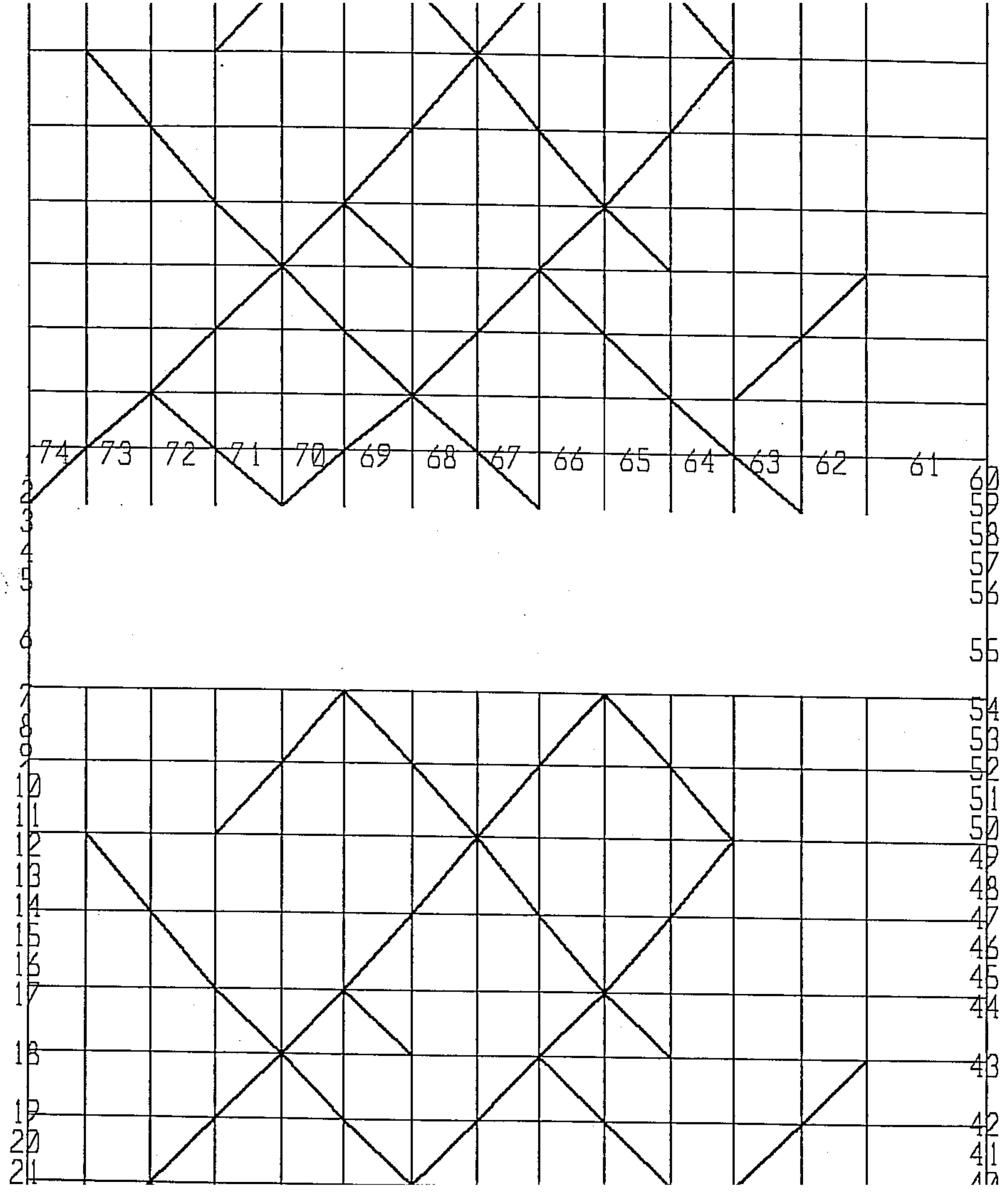


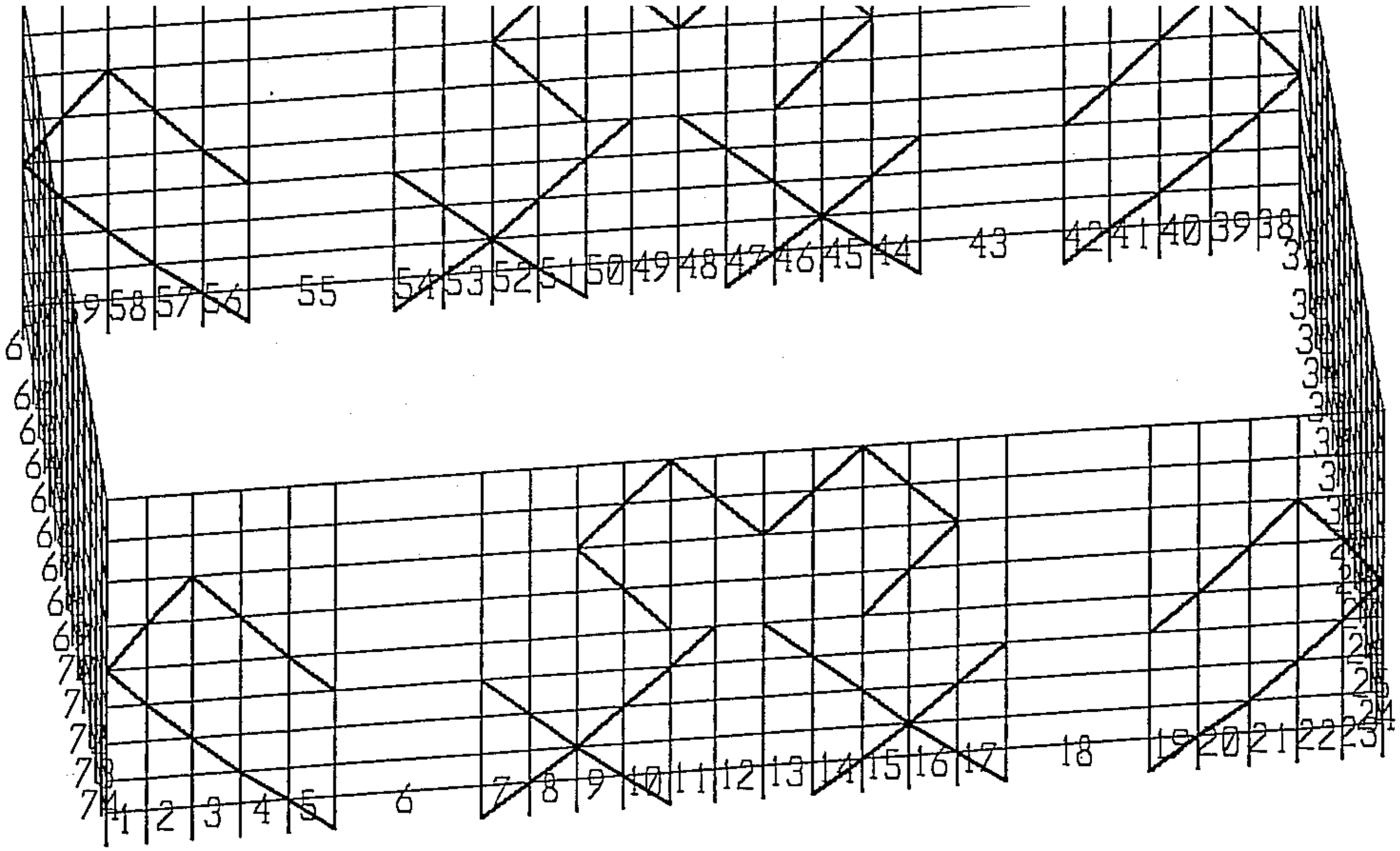
X


1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
X


1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73 74



74 73 72 71 70 69 68 67 66 65 64 63 62 61
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21
60 59 58 57 56 55 54 53 52 51 50 49 48 47 46 45 44 43 42 41


1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58
74

Column Properties

ID	ITYPE	IMAT					
			for User: for Variable:	DMAJ RJ A IMAJ ZMAJ IV ID1	DMIN RIMAJ AMAJ IMIN ZMIN L1 ID2	TF RIMIN AMIN SMAJ RMAJ L2 ID3	TW J SMIN RMIN L3 ID4
1	W10X88	1 Steel		0 1	0 1	0 1	0
2	W10X77	1 Steel		0 1	0 1	0 1	0
3	W10X68	1 Steel		0 1	0 1	0 1	0
4	W10X60	1 Steel		0 1	0 1	0 1	0
5	W10X54	1 Steel		0 1	0 1	0 1	0
6	W10X49	1 Steel		0 1	0 1	0 1	0
7	W10X45	1 Steel		0 1	0 1	0 1	0
8	W10X39	1 Steel		0 1	0 1	0 1	0
9	W10X33	1 Steel		0 1	0 1	0 1	0

Beam Properties

ID	ITYPE	IMAT	DBMAJ	DAMAJ	DMIN	
		TF	TW	RJ	RIMAJ	RIMIN
		for User:	A	AMAJ	AMIN	J
			IMAJ	IMIN	SMAJ	SMIN
			ZMAJ	ZMIN	RMAJ	RMIN
		for Variable:	IV	L1	L2	L3
			ID1	ID2	ID3	ID4
1	W24X55	1 Steel	0	0	0	
		0	0	1	1	1
2	W21X44	1 Steel	0	0	0	
		0	0	1	1	1
3	W18X40	1 Steel	0	0	0	
		0	0	1	1	1
4	W18X35	1 Steel	0	0	0	
		0	0	1	1	1
5	W16X50	1 Steel	0	0	0	
		0	0	1	1	1
6	W16X40	1 Steel	0	0	0	
		0	0	1	1	1
7	W16X36	1 Steel	0	0	0	
		0	0	1	1	1
8	W16X31	1 Steel	0	0	0	
		0	0	1	1	1
9	W16X26	1 Steel	0	0	0	
		0	0	1	1	1
10	W14X30	1 Steel	0	0	0	
		0	0	1	1	1
11	W14X26	1 Steel	0	0	0	
		0	0	1	1	1
12	W14X22	1 Steel	0	0	0	
		0	0	1	1	1
13	W10X33	1 Steel	0	0	0	
		0	0	1	1	1
14	W8X18	1 Steel	0	0	0	
		0	0	1	1	1

Brace Properties

ID	ITYPE	IMAT RJ for User:	DMAJ RIMAJ A IMAJ ZMAJ	DMIN RIMIN AMAJ IMIN ZMIN	TF AMIN SMAJ RMAJ	TW J SMIN RMIN
1	2L4X4X5/16	1 Steel 1	0 1	0 1	0	0
2	2L6X4X1/2	1 Steel 1	0 1	0 1	0	0
3	2L6X4X3/8	1 Steel 1	0 1	0 1	0	0
4	2L4X4X3/8	1 Steel 1	0 1	0 1	0	0
5	2L4X4X5/16	1 Steel 1	0 1	0 1	0	0

Column Properties

ID	ITYPE	IMAT		DMAJ	DMIN	TF	TW
				RJ	RIMAJ	RIMIN	
			for User:	A	AMAJ	AMIN	J
				IMAJ	IMIN	SMAJ	SMIN
				ZMAJ	ZMIN	RMAJ	RMIN
			for Variable:	IV	L1	L2	L3
				ID1	ID2	ID3	ID4
1	W10X88	1 Steel		0 1	0 1	0 1	0
2	W10X77	1 Steel		0 1	0 1	0 1	0
3	W10X68	1 Steel		0 1	0 1	0 1	0
4	W10X60	1 Steel		0 1	0 1	0 1	0
5	W10X54	1 Steel		0 1	0 1	0 1	0
6	W10X49	1 Steel		0 1	0 1	0 1	0
7	W10X45	1 Steel		0 1	0 1	0 1	0
8	W10X39	1 Steel		0 1	0 1	0 1	0
9	W10X33	1 Steel		0 1	0 1	0 1	0

Beam Properties

ID	ITYPE	IMAT	DBMAJ	DAMAJ	DMIN	
		TF	TW	RJ	RIMAJ	RIMIN
		for User:	A	AMAJ	AMIN	J
			IMAJ	IMIN	SMAJ	SMIN
			ZMAJ	ZMIN	RMAJ	RMIN
		for Variable:	IV	L1	L2	L3
			ID1	ID2	ID3	ID4
1	W24X55	1 Steel	0	0	0	
		0	0	1	1	1
2	W21X44	1 Steel	0	0	0	
		0	0	1	1	1
3	W18X40	1 Steel	0	0	0	
		0	0	1	1	1
4	W18X35	1 Steel	0	0	0	
		0	0	1	1	1
5	W16X50	1 Steel	0	0	0	
		0	0	1	1	1
6	W16X40	1 Steel	0	0	0	
		0	0	1	1	1
7	W16X36	1 Steel	0	0	0	
		0	0	1	1	1
8	W16X31	1 Steel	0	0	0	
		0	0	1	1	1
9	W16X26	1 Steel	0	0	0	
		0	0	1	1	1
10	W14X30	1 Steel	0	0	0	
		0	0	1	1	1
11	W14X26	1 Steel	0	0	0	
		0	0	1	1	1
12	W14X22	1 Steel	0	0	0	
		0	0	1	1	1
13	W10X33	1 Steel	0	0	0	
		0	0	1	1	1
14	W8X18	1 Steel	0	0	0	
		0	0	1	1	1

Brace Properties

ID	ITYPE	IMAT RJ for User:	DMAJ RIMAJ A IMAJ ZMAJ	DMIN RIMIN AMAJ IMIN ZMIN	TF AMIN SMAJ RMAJ	TW J SMIN RMIN
1	2L4X4X5/16	1 Steel 1	0 1	0 1	0	0
2	2L6X4X1/2	1 Steel 1	0 1	0 1	0	0
3	2L6X4X3/8	1 Steel 1	0 1	0 1	0	0
4	2L4X4X3/8	1 Steel 1	0 1	0 1	0	0
5	2L4X4X5/16	1 Steel 1	0 1	0 1	0	0

Deficiency	Hazard Level	Implementation & Mitigation
a) Structural (directly related to building structure capacity to support seismic forces):		
Soil Liquefaction Potential	high hazard	soil problems could be solved w/some difficulty
Overstressed foundation	high hazard	pier system could be reinforced w/difficulty
Excessive roof/floor load	high hazard	remove unused/unecessary equipment
Inadequate floor diaphragm	high hazard	reinforce floors w/ in-plane diagonal bracing
Weak flat roofs	high hazard	add roof bracing to strengthen roof diaphragm
Insufficient diagonal bracing	high hazard	provide diagonal bracing to strengthen frame
Poor steel frame connections	high hazard	could be strengthened w/some difficulty
Exterior Wall Panels	medium hazard	panels could be strengthened or replaced
Unbraced Parapets	medium hazard	provide bracing at shorter spans
b) Non-Structural (Non-structural building components or part of other structures in the building)		
Floor-mounted equipment	high hazard	improve equipment anchorage & supports
Roof platforms/catwalks	high hazard	strengthen to avoid vibrations
Duct lateral bracing	medium hazard	strengthen to avoid vibrations
No partition wall seismic joints	medium hazard	could readily be added
No partition wall lateral bracing	medium hazard	could readily be added
No clg compression support	medium hazard	could readily be added
Glazing at entrance	medium hazard	glass could be readily replaced
Staircases & railing	medium hazard	could be readily strengthened
No ceiling seismic joints	medium hazard	could readily be added
No clips on ceiling tiles	low hazard	could readily be added
Light Fixtures Poorly Attached	low hazard	could be readily strengthened
Light fixt./clg separate support	low hazard	could readily be added
Light fixt., stem support	low hazard	could readily be added
Piping, bents	low hazard	provide more anchorage (clamps & supports)
Unseal firewalls	low hazard	could readily be fixed
No pipe flexible joints	low hazard	could readily be added

Oakland, CA 94612
(510) 839-4330

27-Jan-98

ONIZUKA AIR STATION

COST ESTIMATE FOR

A (s.f.) = 170,000

BUILDING 1003

PROJECT CONSTRUCTION BUDGET 14,792,400

Cost/sf 87

a) Structural					
Soil Liquefaction Potential, grout injection		600	CY	500	300,000
Overstressed foundation, reinforce piles		20	EA	10,000	200,000
Excess roof/floor load, remove		1	LS	250,000	250,000
Inadequate floor diaphragm, add bracing		133,000	SF	30	3,990,000
Weak flat roofs, add roof bracing		37,000	SF	25	925,000
Insufficient frame diag. bracing, add bracing		80,200	SF	20	1,604,000
Reinforce steel frame connections		1	LS	950,000	950,000
Partial replacement of heavy ext. wall panels		80,200	SF	40	3,208,000
Unbrace parapets, reinforce		1	LS	50,000	50,000
				Subtotal (a)	11,477,000
b) Non-Structural					
Roof catwalks, reinforce		1	LS	5,000	5,000
No partition wall seismic joints, add joints		1	LS	48,000	48,000
No partition wall lateral bracing, add bracing		1	LS	24,000	24,000
No clg compression support, add support		1	LS	60,000	60,000
No ceiling seismic joints, add joints		1	LS	15,000	15,000
No clips on ceiling tiles, add clips		1	LS	30,000	30,000
Light Fixtures Poorly Attached, reinforce		200	Ea	100	20,000
Light fixt./clg separate support, add support		200	Ea	150	30,000
Roof piping & bents, add anchors		1	LS	15,000	15,000
No pipe flexible joints, add joints		1	LS	16,000	16,000
				Subtotal (b)	263,000
				Subtotal(a+b)	11,740,000
c) Finishing Cost	10%	of	(a+b)		587,000
				Subtotal (a+b+c)	12,327,000
d) Project Cost (A/E, CM, etc)	20%	of	(a+b+c)		2,465,400
TOTAL					[illegible]

APPENDIX D

DATA BASE

A	B	C	D	E	F	G	H

cal Engineering
'ranklin Street
d, CA 94612
39-4330

FINAL EVALUATION REPORT

CONTRACT NUMBER F04689-96-D-Q001

PPRC

Seism

EVALUATION CHART FOR BUILDING 1003

Y CODE	UNIQUE IDENTIFIER	STATE CODE	COUNTY CODE	SEISMICITY	AREA (M2)	NO OF BLDGS	REASON FOR EXCEPT	OCCU
5700	____ 01003 7 / IIIR / 4 67 / 002B	06	085	H	15830	1	E0	

J	K	L	M	N	O	P	Q	R

54

By: F.Mariscal

1003 at Onizuka Air Station

23-Mar-94

L BLDG COD	HIST BLDG CODE	DATE OF CONST	MODEL BDLG TYPE	NO OF STORIES CODE	HIGH RISK CODE	EVAL PROC CODE	SOIL TYPE	FOUNDATION TYPE	OUTCO
Z2	H2	1967	MB04	N04	B	P1	S2	FT2	

T	U	V	W	X	Y	Z	AA	AB

EFIC	NON-STRUCT DEFIC	GEOL/SITE HAZARDS	ADJACENCY PROBLEN	STRUCT COST	NON-STRUCT COST	FINISHING COST	PROJECT COST	SOURCE OF CE CODE	COMME
FS	FN	FG	PA	$11477000	$263000	$587000	$2466000	C3	

APPENDIX E

PROJECT DIRECTORY

CLIENT:

USAF ONIZUKA AIR STATION
750TH CONTRACTING FLIGHT/LGCC
1080 Lockheed Way, Box 039
Sunnyvale, CA 94089-1234
(408) 752-3057 Phone
(408) 752-3862 Fax

Patricia D. Thompson, Contracting Officer
Tony B. Galam, Contract Administrator
Loretta Sung, Chief of Design
Marcella Bailey, Project Engineer
Bertha Roman, Project Engineer
Gene Inserto, Security Escort and Base Photographer

ARCHITECTS / ENGINEERS:

MARISCAL ENGINEERING
1634 Franklin Street
Oakland, CA 94612
(510) 839-4330

Francisco Mariscal, P.E., Principal, Project Engineer and Manager
Nick Pal, P.E., Ph.D., Structural/Geotechnical Engineer
Al Masso, P.E., Ph.D., Civil/Geotechnical Engineer
Toma Goncerenko, P.E., S.E., Structural Engineer
Jose Vallenas, Ph.D., Civil/Structural Engineer
Scott Sparling, E.I.T., Civil/Structural Staff Engineer
Anthony Diaz, Drafter
Carlos Guzman, Drafter
Hilary Mullins, Editor
Maria Elena Lopez, Report Production & Coordination