THE DESIGN AND CONSTRUCTION OF THE BANK OF NEW ZEALAND WELLINGTON, NEW ZEALAND

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1. INTRODUCTION

The new Bank of New Zealand building, will, when completed, form a nucleus at the hub of Wellington's downtown commercial centre. Sited on a strategic corner at the junction of two main thoroughfares, the 30-storey building will be a landmark towering as high again above adjacent new buildings, and three times as high as the older buildings in the locality.

The form of the development results from several years of planning involving an exchange of land between the Bank of New Zealand on the one hand and the Wellington City Council on the other. The existing Bank building is an earthquake risk and will be demolished at the completion of the new building and the empty site made a Civic Park.

The new building will be built on a diagonally opposite corner. It is comprised of (i) a three storey section, 200 ft square which extends beyond the site to the road kerb and is constructed wholly below street level, and (ii) a tower, 112'6" square, rising 331 feet above ground. Open planning at street level makes the whole of the site a public area. (See Fig. 1).

The on-site Branch of the Bank is located at ground lèvel and interconnects with a shopping complex and understreet shopping mall at the first level below ground. A mall connection is made to street level at three points, (i) in the Civic Park, (ii) to the ground floor of the Grand Hotel opposite, and (iii) to the pavement adjacent to Stewart Dawsons diagonally opposite. Full pedestrian circulation will be provided to attract people from street level down to the shopping complex below.

Below the shopping complex, office space is provided to accommodate Branch Bank facilities, and on a further level below, 43 feet below the street, parking is provided for cars. The twenty-six levels above ground are office floors except two floors of plant near the mid-height of the tower. An observation gallery is provided at the top of the building.

Earlier plans had included for a heliport at roof level, but this facility was abandoned largely because wind conditions severely limit the number of operating days.

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2. STRUCTURE

The tower of the building is 112 ft 6 ins. square on plan with an area per floor of 12,656 sq. ft. The service core is placed centrally to give maximum efficiency in floor use and at the same time provide a good arrangement for subdivision.

Preliminary studies of alternative framing arrangements showed that a moment resisting tube or box frame was best, both for economy and for flexibility for running internal services. A ten-column arrangement, 12'-0" on centre, was chosen for the upper levels dividing into four columns each face, 36 ft on centre, from Level 6 down. (See Fig. 2).

Floors are generally 12 ft apart in the upper levels although floor heights increase at the lower levels near ground level. Diagonals are provided at top and bottom of the tower to make the transition between the different column arrangements. They transfer gravity loads directly from upper to lower columns between Levels 6 and 8, and provide a partial top hat truss between Levels 28 and 30 to reduce drift. The perimeter frame is presented externally as the Architectural feature of the building; the trussed levels together with the matched set-back of the glazing at the level of the mid-height plant room giving to the elevations a pleasing 9 x 9 pattern of openings between columns and beams. (Refer Fig. 1).

The floors of the building are composite concrete on steel deck and are supported on castellated beams generally at 12 ft centres. The latter also act compositely with the concrete floor and span 37 ft to the core from the perimeter tube frame. The core frame is simply load bearing. (Fig. 3).

Below Level 3, the tower structure is braced through cross bracing in the floor of Level 3 between the core and the perimeter frame and then by K-bracing between floors in the core framing. This ensures a symmetry of structure and good earthquake performance while providing a rigidity to take the base shear within tolerances which will prevent distress in the lower floors.

3. FOUNDATIONS AND BASEMENT

3.1 Topography and Subsurface Conditions

The site for the building is located over a low promontory which, in earlier times, jutted out into the Wellington Harbour. The promontory has, with the development of Wellington, been landlocked by progressive reclamation (see Fig. 4), so that today it is some three city blocks from the wharves which mark the present shoreline. (See Fig. 5). The subsurface conditions were investigated by drilling eight borings, seven around the perimeter of the site and one centrally within the site. It was found from these borings that the insitu material that marked the old promontory was overlain by only 6'-0" - 8'0" of filling; the promontory itself being comprised of highly weathered to partially weathered Greywacke and Argillite grading to higher strengths with depth.

3.2 Shoring and Underpinning

Three sides of the excavation for the new building are bounded by roadways while on the fourth side an eight storey concrete building adjoins one-half of the site and a four storey brick building adjoins the other; both these buildings are founded on the insitu material below the overlying fills.

Shoring along the street frontages for the 42 ft deep excavation is comprised of twin steel channel, soldier piles at 6'0" centres anchored at three levels as the excavation proceeds by 40 ft to 50 ft long prestressed rock anchors. (See Fig. 20). The distribution of anchors was first calculated to suit an equivalent fluid pressure; and the resulting system then analysed by computer, using wedge theory and anticipated in-mass values of cohesion, c and coefficient of internal friction, ø, for a range of failure wedges. These gave a critical failure angle of 30° to the vertical and a minimum factor of safety of 1.5, which result was considered satisfactory especially as conservative values of c and ø had been used.

Underpinning of the two buildings on the fourth side is effected by using a similar technique; in this case however the soldier piles have a corbel at a level which can be positioned to suit the underside of the adjoining foundation wall. As the excavation proceeded, Freyssinet flat jacks were installed on the corbels; these permitted adjustment for settlement as load is taken up on the piles. Rock anchors are again used to take the transverse rock stresses in the face of the excavation.

All sides of the excavation were monitored by regular surveys on points set on the shoring and on underpinning piles. In general movements occurred immediately after each stage of excavation and then stopped. In the case of the Victoria Street, or seaward, side however more than acceptable movement occurred and this continued long after the excavation was carried out. Accordingly, additional anchors were installed near mid-height. These were grouted and stressed as early as possible. Notwithstanding an additional precaution involving diagonal struts into the site were placed in readiness for loading if necessary. In the event, the additional rock anchors arrested movement and the diagonal struts have not been loaded.

3.3 Foundation Design

At the base of the tower, the building is supported on 20 columns, eight in the structure of the core and twelve in the structure of the perimeter frame. The core columns carry vertical gravity loads and transfer the horizontal base shear to the foundations through the bracing in the lower levels, while the perimeter frame columns carry vertical loads only, resulting from earthquake overturning loads and gravity loads. Even with the transfer the earthquake shear at Level 3 and the consequent overturning forces compression still largely governs the design of the core foundations. The perimeter column foundations, however, develop large tensions in overturning and in an elasto-plastic mode corner columns could, in theory, be subjected to tensions as high as 10,000 kips per column.

To carry these loads, a piled solution has been adopted. Eight foot diameter shafts are provided at each column. These take the horizontal shear from the core in passive pressure and vertical loads by both end bearing and friction along the length of each pile. Corner column piles are belled to take the perimeter tension stresses. Although conventional holding down bolts are used for most of the columns, the corner columns of both the perimeter frame and the core are fixed to their founding piles by means of prestressed cables cast into the piles and anchored over the column base plates. These transfer the tensile forces to the piles and at the same time provide a fuse against loads too great for the pull out strength of the piles.

3.4 Basement Construction

For a fully sealed construction the 42 ft deep basement carries with it a penalty of a high hydraulic uplift. The alternative drained basement construction, however, even though it involves continuous though minimal pumping, is preferable because of a saving in cost of some $200,000. It should be noted that the low level structure is only three storeys high and is not sufficient weight to balance hydraulic uplift unless tied down by piling or by anchoring by other means. Accordingly, the underside of the base slab is drained and the water led away to a pump chamber. It is anticipated that water flow even from the more permeable upper layers can be controlled if not stopped by casting the basement walls against the cut face of the excavation for full height.

3.5 Understreet Mall

The understreet mall extends for almost the total width of the carriageway for the whole of one side of the site and extends lengthwise across the busiest intersection in Wellington. Its construction has involved re-routing of understreet services including sewer and stormwater and the relaying of all other services at new elevations.

Construction was effected by casting the roof during two off-peak holiday seasons. Excavation beneath this roof is now proceeding progressively from below. The periphery to the mall is formed by a row of drilled piles which both serve to support the preformed roof slab and at the same time, retain the lateral earth pressure.

At the inside edge, a system of temporary columns with flat jacks over is used to underpin the slab.

4. SUPERSTRUCTURE CONSTRUCTION

4.1 General

The design for the superstructure was evolved with four basic factors in mind:

(i) The perimeter tube frame was proven in earlier studies to be the most economic solution which also provided flexibility for running services.
(ii) Wellington is subject to windy conditions sufficient to interrupt crane operations on some 20 days per annum. A simple erection method was thus essential.
(iii) Structural steel trades in Wellington region are prone to industrial strife and stoppages. Labour intensive work on site was thus to be avoided.
(iv) The finishing trades should follow the structural steel frame as quickly as possible if advantage of time saving is to be made.

These factors led to a structural steel perimeter frame prefabricated off site with connections made on site by high strength friction grip bolting. Welding was limited to the structural frame below Level 6 and to corner columns of the perimeter frame where tension develops in earthquake.

A steel deck flooring system with steel stairs for ready access were adopted and partitioning and fireproofing were developed in lightweight construction.

4.2 Perimeter Frame

The perimeter tube frame above Level 6 is constructed of column units 2 and 3 storeys high, fabricated from rolled sections and spliced at mid-height between storeys. Beam stubs of half span length are also of rolled section and are fully butt welded to the columns in the fabricator's shop.

The combined beam column units thus formed make a tree structure which is simply connected by friction grip bolts on site. Below Level 6, where the columns are larger, fabricated plate sections were required and splicing here is carried out conventionally by full strength welding on site. Beams are constructed of plated sections, butt welded to the columns also on site.

4.3 The Core Structure

The structure of the core above Level 3 (except for local situations where at Level 17 the floor is not connected directly to the perimeter frame) is subject to gravity loads only and is constructed largely of rolled sections. Columns are machined at interfaces and connected by bolted splices. Beams, too, are simply bolt connected.

Between Levels 1 and 3, bracing comprised of plated sections welded between floors, forms a rigid construction designed to take earthquake shear to the foundations. The members bracing the core at these levels are sized to give very small deflections under load.

4.4 Floor Construction

The main floor spans 37'-4" from the service core structure to the perimeter frame and is comprised of castellated and plain rolled sections at 12 ft centres. A 24 gauge steel deck floor of Bondek pattern spans between beams and is topped by 4½" of dense weight concrete. The steel deck is designed to be composite with the concrete as are the beams spanning to the core. The latter are protected by lightweight fire spray to give the required 2 hour fire rating but the steel deck floor is rated unprotected.

Although there is a small premium for castellating beams, construction has on the whole proven to be very economical. The fact that services can be accommodated through the castellations in the beams is an off-setting cost factor.

5. ANALYSIS AND DESIGN

5.1 Introduction

This section discusses the analysis of the Bank of New Zealand tower for lateral forces, specifically seismic, to the level that governed the design of individual members. Detail consideration of the design of member to member connections is discussed in later sections.

The basic design approach adopted at the start of the design phase is as listed below.

(i) The structure should be designed for the static force levels required by NZS 1900, Chapter 8.
(ii) Elastic simulation of the seismic response should be used to modify the design to remove any undesirable characteristics.
(iii) The resulting structure should be analysed in a collapse mechanism configuration to ensure that all members were capable of sustaining loading consistant with a ductile moment resisting frame.

Finally, the design would be analysed using a programme capable of considering the elastoplastic behaviour of the frame materials under real earthquake motion.

In the analyses discussed in this paper, it was assumed that all lateral loads were resisted by the bare steel frame without the aid of any composite action. Further, the weights of all floors were computed on the basis the actual dead loads of materials used plus 13 lbs/sq.ft for ceilings, services and partitions and 20 lbs/sq.ft. for seismic live load. Except that the seismic live load was increased in plantrooms and other critical areas. The first mode period of the structure on this basis is 3.43 seconds.

5.2 Static Lateral Loading Cases

In early discussions with the Wellington City Council, the design approach previously outlined had been submitted and acceptance obtained in principle. Accordingly, the static earthquake coefficient was taken as 0.0485, this allows for a reduction by 20% of the basic coefficient as provided in NZS 1900, Chapter 8, relevant to Wellington and inclusion of the appropriate allowance for accidental torsions. Under this loading condition, members were sized according to the Australian steel code, CA 1, with inter-storey drifts limited to .005. The actual average computed drift was .0032 with a peak value of .0048 between Levels 4 and 5 on the final structural configuration.

Wind loading cases were computed for storm wind conditions of 1 in 50 year return period. These loadings were derived from a study of the meteorological records available for the Wellington area. The peak gust speed for structural loadings at the top of the building was estimated to be 144 m.p.h. which produces lateral forces of 55 lbs/sq. ft. in accordance with B.S. CP 3: 1970. The new draft loading code DZ 4203 would require a loading of 41 lbs/sq. ft. if normal exposure conditions were assumed. Building drifts during storm wind loading were limited to below .002 in all levels, except from Levels 3 to 6, where the peak was .003.

5.3 Elastic Dynamic Analysis

5.3.1. Choice of Earthquake Record

After discussions internally and then with others, it was decided that El Centro, May 1940, NS, should be considered as the "design event" appropriate to the Bank of New Zealand site. However, so that the design should not become too particularised, the response to other events should be computed and differences noted. Other earthquake records used were as follows:

(i) San Fernando, February 1971. Holiday Inn, NS record.
(ii) Artificial earthquake D1 as derived by Jennings et al [3].
(iii) Artificial earthquake B1, and B2.
(iv) Artificial earthquake A1 and A2.

The response of the structure to some of these records is presented in terms of imposed shear in Fig. 6. The type D1 earthquake resulted in a demand that was lower than that of the code.

5.3.2. Refinement of Structure

As noted above, elastic dynamic analyses were performed in order that undesirable dynamic effects might be removed and the structural system refined. Consequently, a study of the major modifications that resulted from these analyses is most informative.

The transition from 10 to 4 columns is a matter for consideration from the lateral loading aspect. In the first scheme proposed, the bracing in the core up to Level 3 extended as far as Level 6 to improve the flexible and possibly weak lower storeys. Using El Centro 1940 NS as an input record and 5% damping in all eight modes considered, the building base shear obtained was 6600 K, the top storey deflection 22.51 inches, and an average drift of .0063 between Levels 8 and 30. For the same input record and damping, an analysis of a revised structure omitting the bracing in the core between Levels 3 and 6 was performed. In this case, the building base shear obtained was 4800 K, the top storey deflection 21.22 inches, and an average drift of .0055 for all levels above Level 3. This result was much improved and accordingly the modification to the structure was made.

This refinement together with the fine tuning of sizes to the computed dynamic response enabled considerable economies to be made in the design of the structure. This reduced the steel content and enabled the majority of the frames members to become rolled sections. Fabricated members are now required only in the lower levels.

5.3.3. Revised Elastic Design and Buckling Considerations

The dramatic decrease in base shear in simulated response to El Centro achieved by the refinement discussed above, made it practical to consider that the structure be designed to remain elastic. This especially as it was considered that the allowable stress should be 15% greater than the guaranteed minimum yield stress. This is much the same as designing the structure plastically at the guaranteed minimum yield by virtue of the shape factor for I-beams. The elastic approach used made buckling considerations ammenable to treatment by methods contained in present design codes. With the plastic approach, columns are required to be part of a sway prevented structure as the procedure for plastic design of swaying frames is not yet formalised.

With the elastic approach, it was found that in the majority of cases, minor axis buckling governed. In this instance, the effective length factor of all columns about their minor axis was considered to be unity. This was argued on the basis that these did not contribute to the sway stability of the structure and would be braced by the appropriate perimeter frame. Major axis buckling for the perimeter frames was similar to that in ACI 318-71.

This procedure is indicated diagrammatically in Fig. 7 , the moments induced in the column members resisting sway are magnified by a factor

$$\frac{1}{\left(1 - \frac{\Sigma P}{\Sigma P_{cr}}\right)}$$

where ΣP is the sum of the loads on all columns and ΣP_{cr} is the sum of the critical buckling loads of those columns that resist sway loads. Thus it can be seen that gravity loads only produce sway buckling and the overall stability of the structure governs rather than the stability of individual columns.

5.4 Collapse Mechanism Analysis

As previously indicated, the structure was sized so that it would remain essentially elastic under the El Centro earthquake. Nevertheless, recognition was given to the possibility of events which would exceed the elastic capacity of the structure. To this end, a desirable collapse mechanism was formulated as shown in Fig. 8 and the design of elements in which it was desired to inhibit hinging based on a rational analysis of the configuration.

In evaluating the effects of this mechanism configuration, several yield strengths were considered.

(i) Beams in the body of the tower were taken at 1.2 minimum yield.
(ii) The column in which it was desired to suppress the formation of a hinge was taken at minimum yield.
(iii) The beams framing into the column and hence determining the applied moment were taken at 1.4 minimum yield.

The strengths of all members were evaluated plastically on the basis of these stresses and compared.

When considering the design of the paired corner columns, the structure was considered to be in the mechanism mode in both directions. Thus all of the overturning moment is resisted by the axial forces in the extreme columns for the bi-directional attack. Similarly, when the strength of the lower tower floors between Levels 3 and 6 was considered, the shear imposed was taken to be the maximum that the structure above could apply, calculated as the likely maximum shear between Levels 9 and 10 when in the mechanism mode expressed in Fig. 8 occurred.

The piled foundations were also designed for the forces, both tensile and compressive, based on a rational analysis of the mechanism configuration.

5.5 Elastoplastic Analysis

The aim of these analyses was to determine the likely member ductilities and check the overall frame stability during a major earthquake. Since the structure had been designed to remain essentially elastic during the El Centro event, it was decided to perform analyses with both 1.5 x El Centro and the artificial A2 as the basic seismic input.

Studies of other analyses indicated that significant amounts of energy were in fact dissipated by low percentages of critical viscous damping. Accordingly, 1.5 x El Centro was analysed with both 2% and 5% damping. For the purposes of all of the elastoplastic analyses, all yield stresses were considered to be 15% higher than the guaranteed minimum with a strain hardening rate of 5%. The capacities of the columns were considered to be

$$M_{pc} = 1.15\ (1 - \frac{F}{Fy})\ .\ M_p \leq M_p$$

and the effects of buckling on allowable stresses ignored.

Overall stability of the structure was assured by including the P delta effects in the analyses.

Figs. 9 and 10 compare the distribution of total input energy for the 2% and 5% damping runs using 1.5 x El Centro. It is noteworthy that the total input energy is lower in the 2% damping case by virtue of the greater hinging within the structure and consequent changes in the instantaneous stiffness of the system allowing it to escape excitation to some degree.

Fig. 11 compares the maximum storey shears imposed on the structure during the elastoplastic analysis with the same input data and elastic response assumed. For comparison, the mechanism shear capacity as computed from Fig. 8 using the same yield stresses as the elastoplastic analyses is also shown. The most likely explanation as to the ability of the frame to resist shears greater than those expected from the collapse mechanism is the departure of the point of inflection from the mid-storey height. In the extreme case, there is the possibility of columns being in single curvature.

The table below indicates the maximum ductility demand for both beams and columns in the various analyses.

TABLE 1.

MAXIMUM BEAM AND COLUMN DUCTILITY DEMAND

	1.5 x El Centro 2% damping	1.5 x El Centro 5% damping	A2 2% damping
Beams	3.37	3.08	4.92
Columns	3.47	2.54	1.60

where for all elements ductility is defined as total rotation divided by rotation at yield. The maximum beam ductility demand occurs near the mid height region of the tower while for columns the maximum is at Level 8. The high column ductility demands for the El Centro runs appear to be attributable to a sudden increase in axial load when the section is already resisting beam capacity moments.

One of the basic attitudes that was adopted during the selection of member sizes was that all members should be as closely designed as was possible. Beams, however, were held at a constant size on any one level and if necessary to match the typical beam moment for the level, allowed to be overstressed in the worst location when designing by the procedure outlined in 6.3.3. The basis for this approach was an attempt to reduce the peak ductility and spread the deformation as evenly as possible throughout the structure. The success of this approach is self-evident from the table presented below.

TABLE 2.

BEAM DUCTILITY STATISTICS AT LEVEL OF PEAK DUCTILITY

Earthquake	Level of Peak	Peak	Mean	Minimum
El Centro 2%	22	3.37	2.78	1.92
El Centro 5%	22	3.08	2.58	1.74
A2 2%	21	4.92	4.10	3.27

It is gratifying to note that for both

of the increased El Centro runs and the artificial A2, the elastoplastic analyses performed gave no indication of potential instability or premature failure.

5.6 Discussion and Conclusions

One of the factors to be considered in the design of structures like the Bank of New Zealand tower is the relevance of any seismic loading code. While NZS 1900, Chapter 8, specifically excludes all structures over 200 feet high, it is still regarded as a binding requirement by most designers in New Zealand for all high rise building frames. Initially a code loading coefficient of 0.061 was adopted for the basic requirement, but it was found from elastic simulation of the response that the "true" response to all of the events discussed in 6.3.1 was better when the structure complied only with the requirements of 80% of the code.

As previously stated, the building was designed to remain essentially elastic during the El Centro event. This required a design to a base shear of 4700K, the factor of safety involved in a code elastic analysis is

$$\frac{\text{expected yield}}{\text{allowable stress}} \times \text{shape factor}$$

$$= \frac{1.15fy}{.67 \times 1.33 \quad fy} \times 1.18 = 1.52$$

This factor elevates the 80% code base shear to 1.52 x 2500 = 3800 K, which is 79% of the El Centro base shear. Thus in essence the strength characteristics of the Bank of New Zealand tower are greater than those required by NZS 1900, Chapter 8.

In terms of drift requirements the structure as designed only just meets the code requirements under the allowable 80% coefficient. However, this basic statement is misleading and a better picture is obtained if actual drifts to say the El Centro event are considered. For the Bank of New Zealand tower, the actual elastic computed drift under dynamic loading is .0087. This is equivalent to 0.003 at code loading for framed structures of 8 to 10 storeys if the three times factor on deflection is included.

Another basic difference of the effect of major earthquakes on high rise structures from those of fewer levels is the number of reversals imposed. For a structure of 3 seconds period like the Bank of New Zealand, the El Centro event only produces two major excursions during the time of excitation whereas for a lower structure, this figure would be doubled or even trebled.

Another item for discussion is the basic assumption of completely resisting all loads by the bare steel frame alone, in reality the beams will act compositely with the topping to the "Bondek" slab and this significantly stiffens the building. It was for this reason that a relatively low damping of 2% was considered in the elastoplastic case on the assumption that although the stiffer composite structure would attract more load the damping would probably increase sufficiently to offset the effect.

Having designed a structure of the size of the Bank of New Zealand tower, it would appear that a basic or code coefficient has three effects:

1. It produces a strength and a stiffness such that the building may be subjected to minor earthquakes without damage, structural or non-structural.
2. For major seismic disturbances it serves the purpose of producing a residual strength and stiffness sufficient to prevent the onset of collapse through instability.
3. It reduces the ductility ratios to a level that can be achieved by the materials of the construction.

Provided that this basic coefficient is sufficient to achieve the aims stated above, then attention to detail remains the item of paramount importance. It is more important to achieve a stable structure that will not fail prematurely at any location than to arbitrarily increase the coefficient.

6. DESIGN

6.1 Introduction

Before designing the individual members, it was necessary to choose a code that would be applicable to the building. It was noted that the New Zealand steel code is not relevant for the design and detailing of modern multistorey structures for seismic conditions. However, a lead is given in the current New Zealand code 3101P which refers to the Structural Engineers Association of California (SEAOC) seismic code; this code has a section on structural steel.

Although SEAOC in turn refers to the American Institute of Steel Construction (AISC) specification, the relevant section in this code on plastic design has been based on monotonic loading. Notwithstanding, it was this code that was considered the best base available as much of the research on cyclic loading has been founded on the parameters set out in the AISC specification for plastic design. Early in the design, the decision was made to use structural steel, and not concrete. This decision was made in recognition of the better performance of structural steel in the plastic range and that materials proven capacity to absorb energy.

For this section, the main seismic resisting elements of the perimeter frame will be referred to as main members. The non-seismic core columns and floor internal beams shall be termed secondary members.

6.2 Steel Specification

The choice of the particular steel to be used was made on both an economic and a performance basis. Once the best method of fabrication (combination of welding and friction grip bolting) had been determined, then the suitable steel was decided to suit these methods. The following items were also considered in the choice of steel:

(i) Uniform composition.
(ii) A maximum limit on carbon equivalent (for weldability).
(iii) Freedom from elements likely to cause hot cracking.
(iv) A steel that is relatively clean, i.e. free of large laminations and inclusions.

As discussed above, greater flexibility of the structure resulted in a reduced inertial response to earthquake. To achieve this greater flexibility a higher strength steel was chosen for the beam for much of the structure. Additionally, for the column sections where buckling is a criterion the general relationship between yeild stress, area, radius of gyration, and cost will always favour the use of high strength steel. [8] The analysis of these factors lead to the choice of steel which had a minimum yield of 50 kips/ sq. ins and the following steels were considered to be relevant.

ASTM A36 and A441
ASA 186, 1971, grades 250 and 350
BS 4360, 43A and 50B.

Each of these steel specifications gives a relatively tight control on carbon equivalent which affects hardness, on sulphur and phosphorus with their influence on hot cracking and elongation and minimum yield strength. They satisfied our requirements (i), (ii) and (iii) above, but Item (iv) was not satisfied by commercially available building steel made to these specifications without cost implications. Also there is the lingering doubt that no matter how tight the specification of the steel, it will not necessarily prevent problems during fabrication. It should be emphasised that ultrasonic scanning of the plate before fabrication and specification of a higher grade of notch ductility will not necessarily be a universal panacea for all fabrication ills. Most steels can be made faulty by bad welding and fabrication or poor design details. However, it is well established that the greater the thickness of the steel, the greater the chance of problems arising. For this reason it was decided to qualify the above steels, viz. all 3" plate for the fabricated columns should be grade LO and all sections and plate 2 inches and over in thickness should be fully killed.

In the writer's opinion, such phenomena as brittle fracture and fatigue failure are caused largely by poor design detail or faulty fabrication; their incidence can only be reduced by steel selection but they can certainly not be avoided entirely in this manner.

6.3 Member Design

6.3.1. Main Members

As stated in Section 6.1 above, the design criteria used for the main members was the plastic design section of the AISC, Part 2. It was decided that the site welding would be kept to a minimum and that where possible joints would be made on site by the use of high strength friction grip bolts. In fact, the jointing criteria adopted was

(i) beams would be welded to columns in the fabricator's shop.
(ii) alternate columns would have joints at different levels.
(iii) the jointing of beam to beam would be by high strength friction grip bolts.
(iv) column to column splices would be made by partial penetration butt welds in the flanges and by sufficient bolts in the webs to develop the ultimate shear of column in a storey height, i.e. beam capacity hinging at the top and column hinge forming at the bottom. The exception to this requirement was the corner and adjacent column which have full strength welds because of the significant tensions which can develop in the mechanism condition.

Typical connections are shown in Fig. 12. For item (i) the joint adopted was that which had shown the greatest energy absorption capacity in the tests carried out by Popov [9]. This joint has full strength butt welds for flanges and web.

For item (iii), bolts are in double shear with web plates either side of the web.

The holes in the main beams have been drilled a quarter inch over size and the grade of bolt used is ASTM 325 friction grip type. It should be added that the joint has been given a moment capacity by adding reinforcing bars in the concrete slab immediately above the joint. The moment provided for is the greatest moment occurring as computed from the El Centro elastic computer run. All beams have ¾" diameter shear studs at 12 inch centres. The minimum design requirements for the studs to main beams was those necessary to transfer the floor shear and also to provide a restraint for columns about the YY axis.

At Level 5, because the floor slab is omitted, vertical plates have been welded to the beam flanges to minimise buckling. The member analysed has a buckling capacity such that hinges can still form at the beam ends.

At Level 3 where the building shear is transferred to the core the beams have been designed compositely (10) in order that hinges may form at their ends and the significant horizontal loads are also provided for. The diagonals have been designed for the resultant axial loads. Joints are given the capacity to develop the full member capacity.

6.3.2. Secondary Members

(i) Flooring

The floor in the greater area of the building is a composite metal deck, comprising 4½" concrete and 24 gauge Bondek. The floor in all cases is continuous over a minimum of two spans and at the theoretical end support negative reinforcement has been added to give a capacity of $WL^2/24$. If this criteria is satisfied, the floor could achieve a 2 hour fire rating without protection. As an added precaution too, the negative reinforcement which is an oblong mesh, was made continuous and allowed to drape down to the top of the ribs of the

Bondek at mid-span. The positive moment region was then checked for a loading of 1.25 (DL + 0.33LL) and the metal deck ignored. This then became our design criteria for the slabs. In later discussion with Lysaght of Australia, who had then completed tests for the fire rating, the recommendation was made to design the slab on the reduced depth and adopt a working stress at a level slightly below yield. This particular floor now has New Zealand Standard approval for a fire rating of two hours.

In general all the main tower beams are castellated; the beams being made from differing weight sections. The heavier section has been provided in the bottom half of the beam and then the total section designed to act compositely with the floor slab. The design criteria used for the composite beams with metal deck was that recommended by J. W. Fisher of Lehigh University [11]. Main supporting secondary beams were of solid section and designed to maintain a uniform overall depth of floor at a nominal 24 inches below the 4½" slab. To achieve this, the yield stress for these members was increased to 50 kips/sq. ins. Additional investigation was carried out for the floors to check vibration and to keep perception at a tolerable level. Although not strictly comparable the work of Lenzen [12] was used as a guide.

As mentioned previously, the castellated beams were chosen to facilitate the running of services but in many cases additional openings or an increase in opening size was required. Using the design rules of Redwood [13], which is a plastic criterion, significant savings have been achieved in the amount of reinforcing required for penetrations and in many cases no reinforcing at all has been required. Where a great number of openings occurred in a beam, a closer look has been made of deflection, taking into account shear and rotation at a penetration as well as the normal flexural calculations. For composite cases, long-term deflection and shrinkage have been calculated.

It should be added that no penetrations were allowed in the hinging regions of the main structure and penetrations away from plastic regions were required to be symmetrically placed.

(ii) <u>Columns</u>

All core columns have a minimum yield strength of 50 kips/sq. in. and have been designed with an equivalent length factor K of unity. An analysis using the approach of Yura [6] was made to ensure that the main frame could indeed provide sufficient bracing to validate this assumption. At splices the direct load was transferred by machining the matching surfaces and providing sufficient high strength friction grip bolts to cover in inherent eccentricity in the column formulae and for residual moment from beam eccentricity. Beams framing into the column were designed with connections of sufficient capacity to brace the columns as well as transfer their own loads. At levels 7 and 18, where the floors do not carry out to the perimeter frames, the core columns have been designed to carry the floor seismic loads to the floors immediately above and below.

6.4 <u>Joint Design</u>

As previously stated, the criteria for the design of members required to resist substantial inelastic rotations was that given in Part 2, AISC Specification and which was formulated and documented at Lehigh University [10]. A minimum yield stress of 50 kips/sq. ins was used in the member sizing and the criteria for compact sections [10] was taken to be that of minimum yield stress rather than the average values. Since fabrication values listed on member and plate certificates has shown the average yield to be approximately 15% above minimum.

For the design of individual columns versus beams the criteria has been that stated in 5.4.

The column was checked for the following.

(i) Web crippling $W > \left[\dfrac{Abf}{tb + 5Kc}\right] \dfrac{Yb}{Yc}$ (10)

W = column web thickness
Abf = area beam flange
Tb = beam flange thickness
Kc = dimension from outside of column flange to bottom of root radius
Yb = Yield stress of columns
Yc = Yield stress of the column

(ii) Tension distortion of the flange $Tc > 0.4 \sqrt{Abf \dfrac{Yb}{Yc}}$ (10)

Tc = column thickness

Horizontal stiffeners were provided for the maximum of either (i) or (ii). The stiffeners were detailed one inch wider than calculated and bevelled at the column root radius.

In the initial design of the joints for shear, it had been intended to use diagonal stiffeners in columns which did not support secondary beams and doubler plates in columns which did. The use of doubler plates facilitated the secondary beam connection.

(iii) Joint shear. The web thickness W of the column within the joint to be

$$\frac{\sqrt{3}\left[\dfrac{M1}{db} + \dfrac{M2}{db} - Vc\right]}{Fy\ dc\left[1 - \dfrac{P}{Py}^{2}\right]^{\frac{1}{2}}} \qquad (13)$$

db = beam depth
Dc = column depth
Vc = column shear
P = axial load
Py = squash load of the column
M1 & M2 = beam moments
Fy = minimum yield stress of the column.

For the horizontal stiffeners M1 and M2 are based on a value of 40% over minimum yield plastic moment.

It should be noted that the more recent

Lehigh investigations [13] have shown that axial load plays a significant part in the capacity of the beam column joint when high shears are imposed.

(iv) Web Buckling

$$\text{For } \frac{P}{Py} \leq 0.27 \quad \frac{d}{t} \leq \frac{412}{\sqrt{Fy}} \left(1 - 1.4 \frac{P}{Py} \right)$$

$$\frac{P}{Py} > 0.27 \quad \frac{d}{t} \leq \frac{257}{\sqrt{Fy}}$$

Because the main column section chosen for the tower, a 27W177 with Fy = 50 kips/ sq. ins has web width to thickness ratios in excess of the above it had been intended to either carry the doubler plates 24 inches above the joint or provide an alternative form of web stiffening. However after further consideration it was decided to carry the doubler plates 5" above and below the joint and check by test. There was in fact no definitive source that could be used as a guide to check buckling under the combination of high axial load, shear and moment.

6.5 Testing of Sample Beam - Column Joint

The plastic criteria used for sizing members and the beam column joint, had been derived from monotonic loading (refer 6.1). The cyclic work by Popov et al [9] had been mainly carried out on members which are significantly smaller than would occur in multistorey structures. However, the work of Popov was used as the basis upon which the design joint was made.

After reviewing the elastoplastic 1.5 El Centro computer run with 2% damping, a beam and column unit at Level 10 was selected which was felt would be representative for the structure.

The members were	column	27W177
	beam	27W102
	Horizontal stiffeners	4" x ½"
	½" doubler plates	each side of the web

For the first of the three tests, the doubler plates were carried 24" past the flange stiffeners. This was reduced in the second test to 3" past the flange stiffeners and for the final successful test 5" past the flange stiffeners. (See Figs. 16, 17 and 18.)

All steel had a minimum yield of 50 kips / sq. ins. The welding of the beam to column was detailed as double V butt welds, the doubler plates to column web: full strength butt welds, the horizontal stiffeners to column flange: ½" fillet welds and stiffeners to the column web: ¼" fillet welds. The electrode used for welding was specified as low hydrogen classification E70XX. The axial load and the shears in the beam sufficient to form plastic hinges at the column face.

The loading pattern used was that described by Hanson and Connor [6]. The tested joint is shown in Fig. 15.

The load was applied to the test column by means of 6 - 2" dia. Ultimo bars and anchored off. Load was then applied to the beams by alternating jacks. The test was carried out at the School of Engineering, Auckland University under the direction of Professor R. Shepherd.

A full description will not be given of the test in this paper as this is the subject of an unpublished paper [16] by Professor R. Shepherd and K. C. F. Spring.

A brief summary of the three tests is as follows. For the first two tests, the design loading was not achieved. In the first test welds were made incorrectly, and not to specification and in consequence an early failure resulted.

For the second test, the beam sections were discarded, but the same column section re-used. However, the welds were once again defective, in that they had been very much over-reinforced. Failure this time occurred at the junction of column flange and web and extended through the weld connecting the doubler plates and vertically through the column flange. It was concluded that the initial working of the first test, together with the excessive welding in the second stage fabrication locked high welding stresses beyond yield into the junction and these resulted in the premature failure. In each of these two tests, the doubler plates stopped at the root radius of the column web and the butt weld was made to fill the gap between the doubler plate and column flange.

Following the second failure, Professor G. Ferguson was engaged to inspect and report from a metallurgical point of view and give his conclusions on the failure. Residual stresses were highlighted by him and in subsequent re-assessment of the design, welds were detailed to be kept to a minimum such that they had the capacity to develop only the pertinent section they were connecting. The third test reached the required loads successfully (See Figs. 13 and 14).

We would conclude from the test that over welding can be as adverse an affect as under welding on member performance. Also it is essential that the inspection be meticulous and thorough.

6.6 Welding Standards and Control

The code adopted for welding and Inspection was AWSD1.0-69 and its subsequent amendments. At an early stage, Victorian Welding Supervision Pty. Ltd. (VWS) were engaged for the fabrication and welding inspection. During the detailing of the structural steel, root gaps and weld angles had been determined in accordance with ASWD1.0-69. These were modified after discussion with the inspection authority and subsequent procedure tests by the fabricator. Significant savings were achieved in this way in weld sizes.

Ultrasonic non-destructive testing was specified for all beam column joints, and joints of significant tensions. These to be carried out after fabrication. Prescanning of the plates prior to fabrication had, however, not been specified. This was instructed following the testing of the prototype joint and is now being carried out. The requirement may be relaxed once satisfactory results are achieved in the shop.

Following the welding of a joint, a macro test is required and after 48 hours the joint is ultrasonically scanned. It has been found that cracking of a weld is time dependent, and for this reason final inspection is delayed.

The presence of laminations or significant percentages of inclusions at a critical region is a concern for the designer, but unless a Type A or Type B steel is specified (i.e. special steels used for pressure vessels) then commercial building steels will always have some faults. In consequence, the inspector and fabricator were advised to ensure that the steel at a critical region is as clean as possible. It is essential that the fabricator be aware of the possible problems with "unclean" steel and that should faults occur then remedial measures be such that an effective repair be made.

No amount of prescanning will guard against the phenomena of lamellar tearing. Control for such an occurrence is better by sound detailing and fabrication rather than by material selection procedures.

7. GENERAL FEATURES

7.1 Cladding and Glazing

A series of alternative cladding treatments was investigated until finally precast concrete was adopted. Cladding units are 12'-0" square and are separated at the centre line of both columns and beams. The 9'-0" square section in the centre of the panels is glazed with twin lites of glass up to 12 mm thick set in aluminium frames.

At the corners of the building, a movement of the panels up to 1½ inches is provided for, and individual cladding panels are articulated to accommodate distortion of the frame between floors. (See Fig. 19).

The whole facade must combine simple practical construction with insulation and fire separation while at the same time producing a weathertight curtain. A full scale mock-up of a typical bay is being constructed and will be tested by racking to prove fixings and joint sealing tolerances.

In a structure of the type developed for the subject building considerable lateral movement must be allowed to occur in earthquake if the earthquake response is to be kept to a minimum. Internal core partitions, cladding and glazing must all be designed to accommodate this movement.

Lateral drift between floors has been limited to 0.0017 times inter-floor distance for storm wind conditions or to .010 times inter-floor distance for earthquake. That is, to ¼ inch and 1½ inches respectively.

7.2 Partitions

All internal partitions are of lightweight construction. A nominal degree of freedom is provided at intersections with the frame but in a very severe earthquake some distortion of partitions and thus damage to finishes may occur. Such damage however is comparatively easy to repair.

7.3 Seismic Gap at Level 4

Because of the lack of symmetry of the construction of floors around the tower below Level 4, it was decided to transfer earthquake forces into the core and thence to the foundations through core bracing below Level 3. The tower is thus able to move free of the peripheral structure of the basement from Level 3 up. This movement is provided for at the junction of the plaza level with the tower at Level 4. The maximum computer value of movement at this level is 1¼" and is provided for in the detailing of the panel of glazing at the perimeter of the tower.

7.4 Fire Protection

The New Zealand fire code requires the following fire ratings:

Floors	1½ hours
Beams supporting concrete floors	2 hours
Columns	2 hours
Walls to stairs and lift shafts	2 hours

In addition, for a building higher than 150 ft an approved automatic sprinkler system must be installed in all areas.

In the subject building, the steel deck floor is rated unprotected for 2 hours while beams and columns are fire protected with lightweight spray to give the 2 hour rating; walls are constructed of Gibraltar board on metal stud. Authority approval for the lightweight spray was only given after exhaustive submissions and then providing a detailed specification and a very full supervision of the application was established. This attitude of the Authorities was frustrating but understandable considering the abuses of lightweight spray in other countries and the reports of fire losses due to poor application or damage to protective sprays by following trades.

7.5 Wind Tests

Wellington is a windy city with recorded 3 second gust velocities of 123 m.p.h. at sea level. Although partially protected by a considerable region of built-up area to the windward in the direction of the most severe winds, and by a topography in other predominant wind directions, it is known that excessive windy conditions occur in the downtown Wellington area. Accordingly model tests were carried out to determine wind effects at and near ground level. In addition to these environmental tests, quantitative velocity measurements were made around the 1/16" scale model. These were necessary for two reasons:

(a) to determine maximum local pressure for window design;
(b) to determine back pressure at points of exhaust and intake to the air

conditioning system.

7.6 Air Conditioning

The building is designed so that all normally occupied areas will be fully air conditioned. The system used is comprised of a combination of "all air" for the centre zone and "induction" for the perimeter, with central zone terminal equipment mounted in the ceiling spaces, and induction units are floor-mounted adjacent to windows. The system serving the tower down to Level 8 will be based on plant located in the central plant room (Levels 17 and 18) while areas from Level 7 downwards will be served by separate air conditioning and ventilation plants provided locally to suit the functional requirements.

7.7 Vertical Transportation

The main passenger lifts will comprise 4 high rise lifts each of 2500 lbs capacity travelling at 100 ft per minute and 4 low rise lifts each of 2500 lbs capacity travelling at 700 ft per minute. The high rise lifts will have stops at Levels 4, 7, 16, 19-28 inclusive, while the low rise lifts will have stops from Levels 1 to 16 only. Other lifts are provided for service goods and an additional lift is provided to serve the executive suite. One of the latter will be available for use by firemen.

For public transportation between Branch Bank, shopping complex and the Plaza levels, six large capacity escalators are provided.

8. THE DESIGN TEAM

The Architects for the project are Stephenson & Turner, while the structural Engineers are the writer's firm, Brickell, Moss, Rankine & Hill.

Early in the development of the project, submissions were sought from four Contractors for a selection to be made on the basis of typical bulk rates, profit margin, and upon the proposed project team. After receiving submissions, Messrs Civil & Civic Pty. Ltd. were selected as Design Consultants for the project - with a provisor that they could become the Main Contractor if satisfactory construction costs were negotiated. The selection of a services Building Consultants was made in a similar way.

The overall contract is divided into four phases

(i) Demolition
(ii) Site Works
(iii) Structural Steel Framing
(iv) Concrete and Building Works.

The arrangement has proven a successful one, with interplay between designer and builder throughout the detailed development. In retrospect, it is difficult to imagine how so much could have been achieved in so short a time without such an arrangement.

9. THE CONSTRUCTION CONTRACT

One major contract that was separated and awarded early in the programme was the structural steel contract. For this contract, a lead time of eight months was required for ordering and delivery of steel, for completion of shop drawings and for fabrication of sufficient steel in time to meet the programme for the erection on site. A concurrent contract was also awarded for site work, mall construction and piling to enable work to proceed while documentation of the total building was being completed.

At the completion of documentation, an overall contract was negotiated and awarded to Civil & Civic Construction Co. and the structural steel contract absorbed as a nominated sub-contract within this contract.

10. FINAL COMMENT

The new Bank building in Wellington will, when completed, be the tallest building so far constructed in New Zealand. Its total height from basement to roof is 374 feet.

The design incorporated a sophisticated analysis involving real time earthquake motion to achieve an economic structure tuned to suit the earthquake response. A safer building resulted which was also competitive in cost and in end return for investment.

11. ACKNOWLEDGEMENTS

We would like to thank the many people who have given assistance on this project. In particular the following organisations:

Computer Analysis: Albert C. Martin & Associats, Los Angeles.

Prototype Testing: School of Engineering, University of Auckland.

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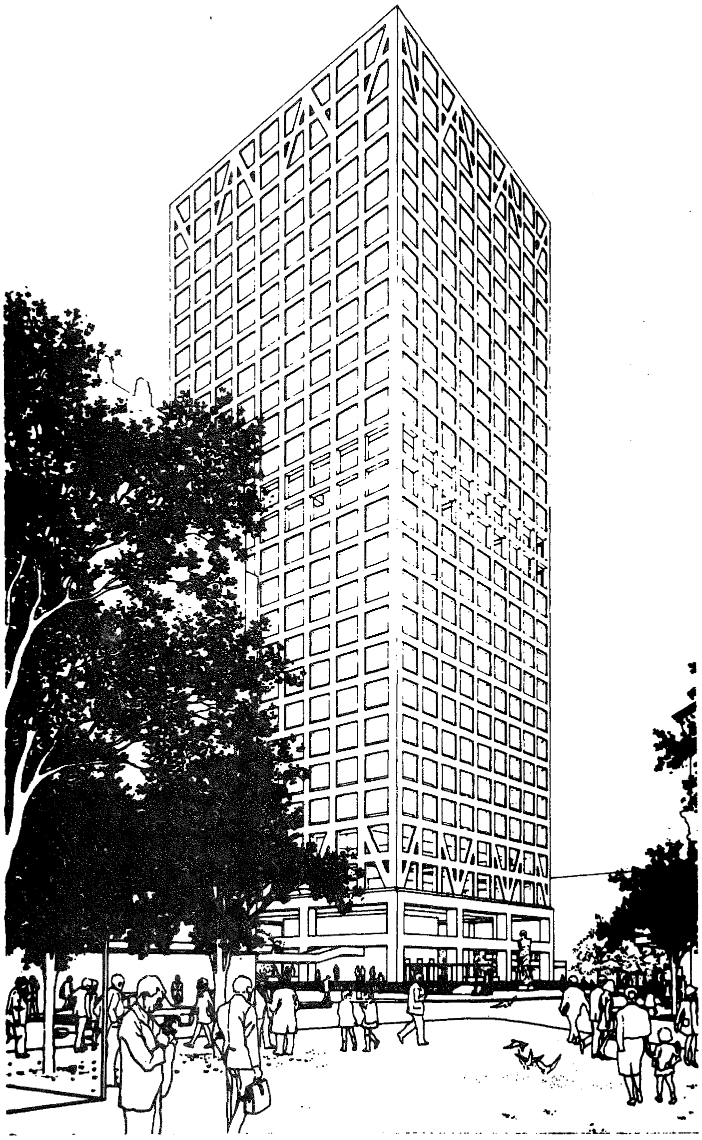
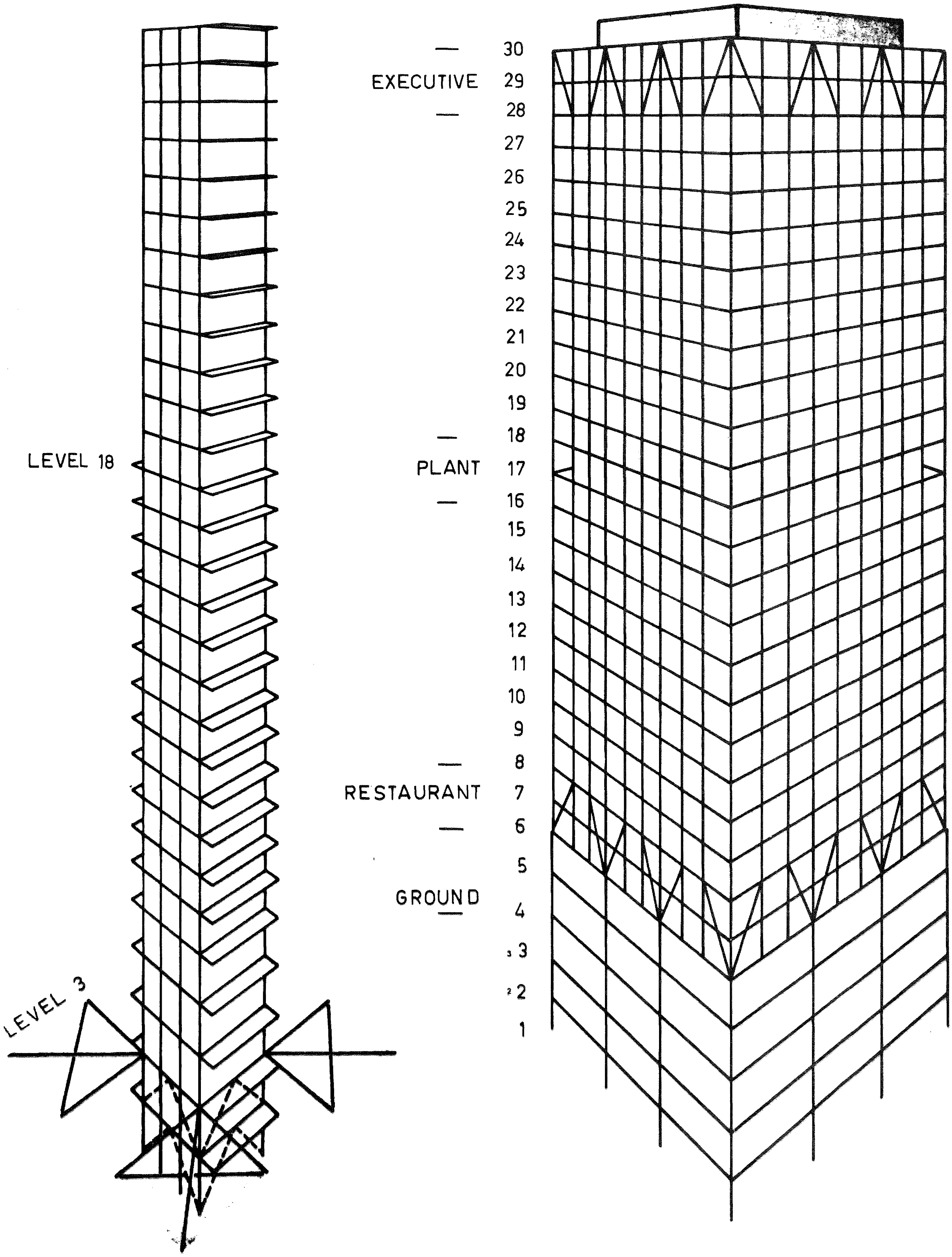


FIGURE 1:


LEVEL 18
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EXECUTIVE 29
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PLANT 17
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RESTAURANT 7
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GROUND 4
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FIGURE 2:

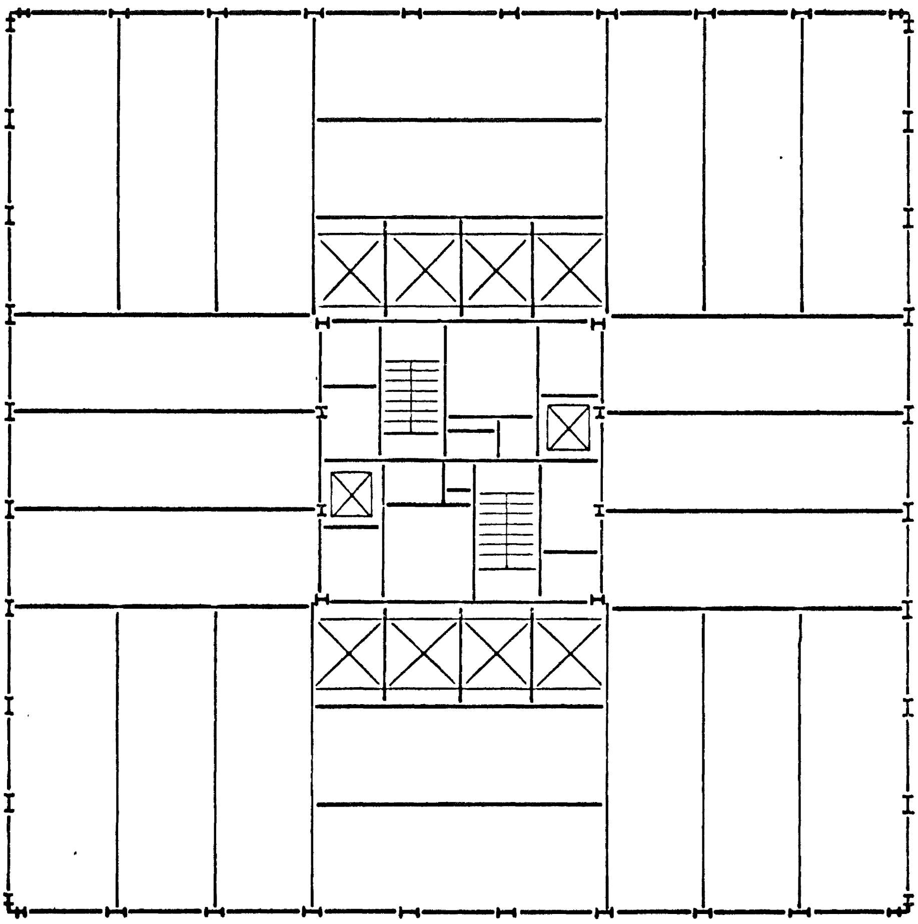


FIGURE 3: TYPICAL FLOOR PLAN

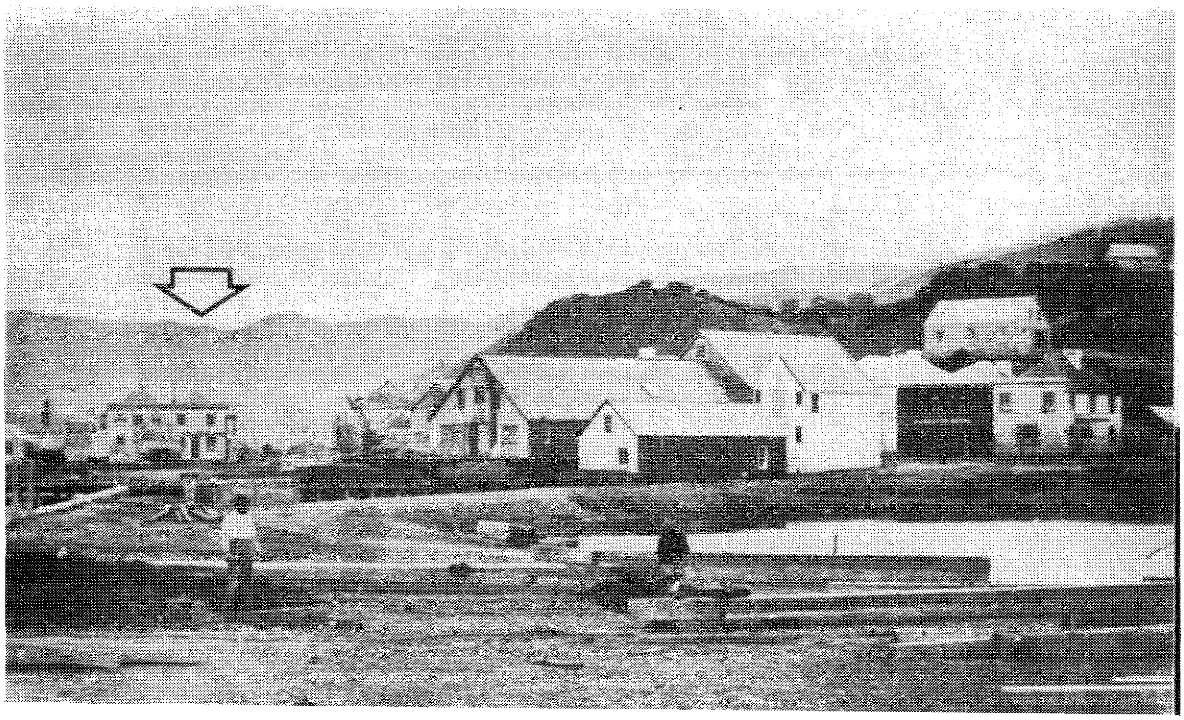
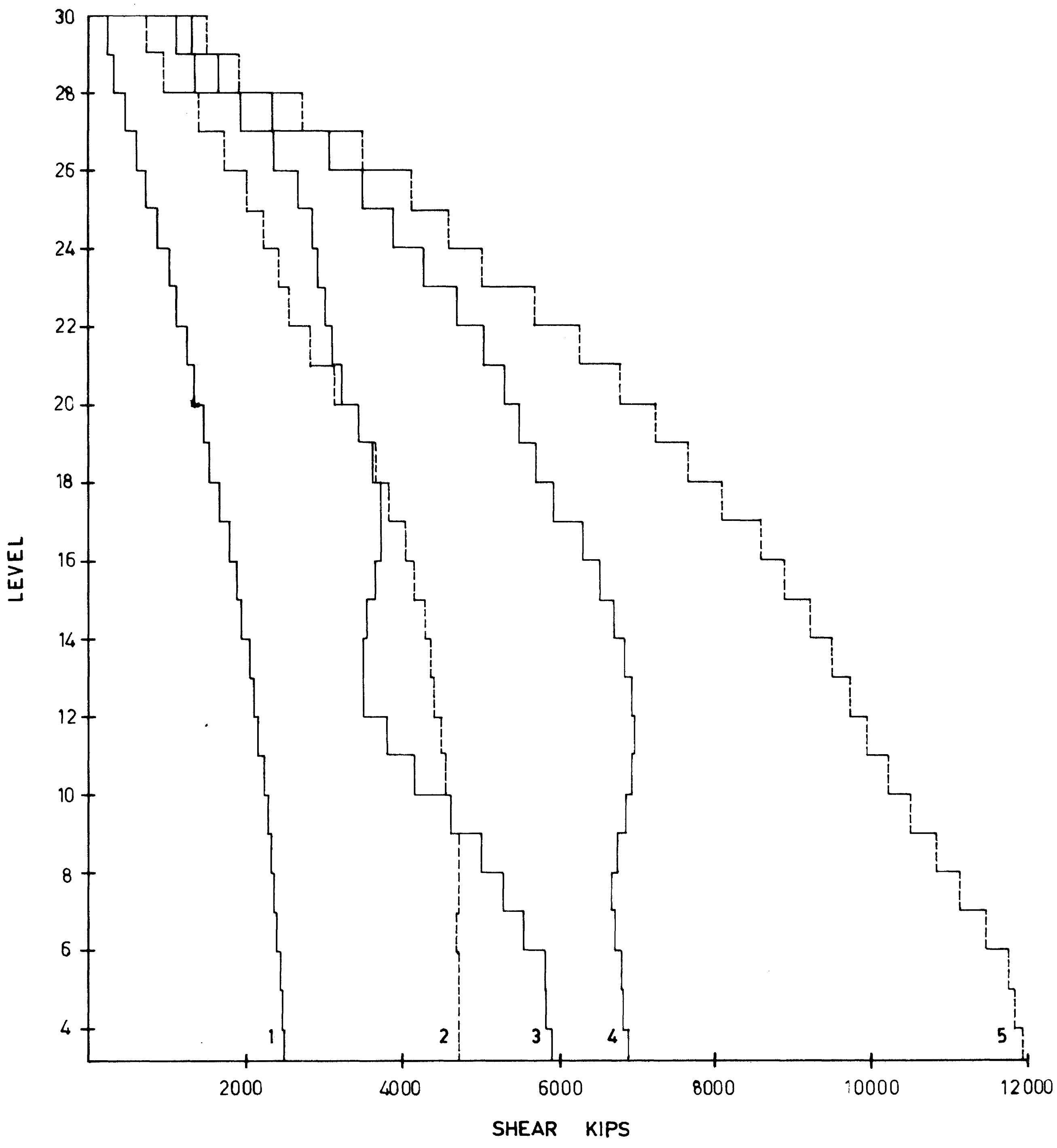


FIGURE 4: (PHOTOGRAPH THROUGH COURTESY OF THE
WELLINGTON PUBLIC LIBRARY

FIGURE 5:



1 80% NZSS 1900 CHAPT. 8

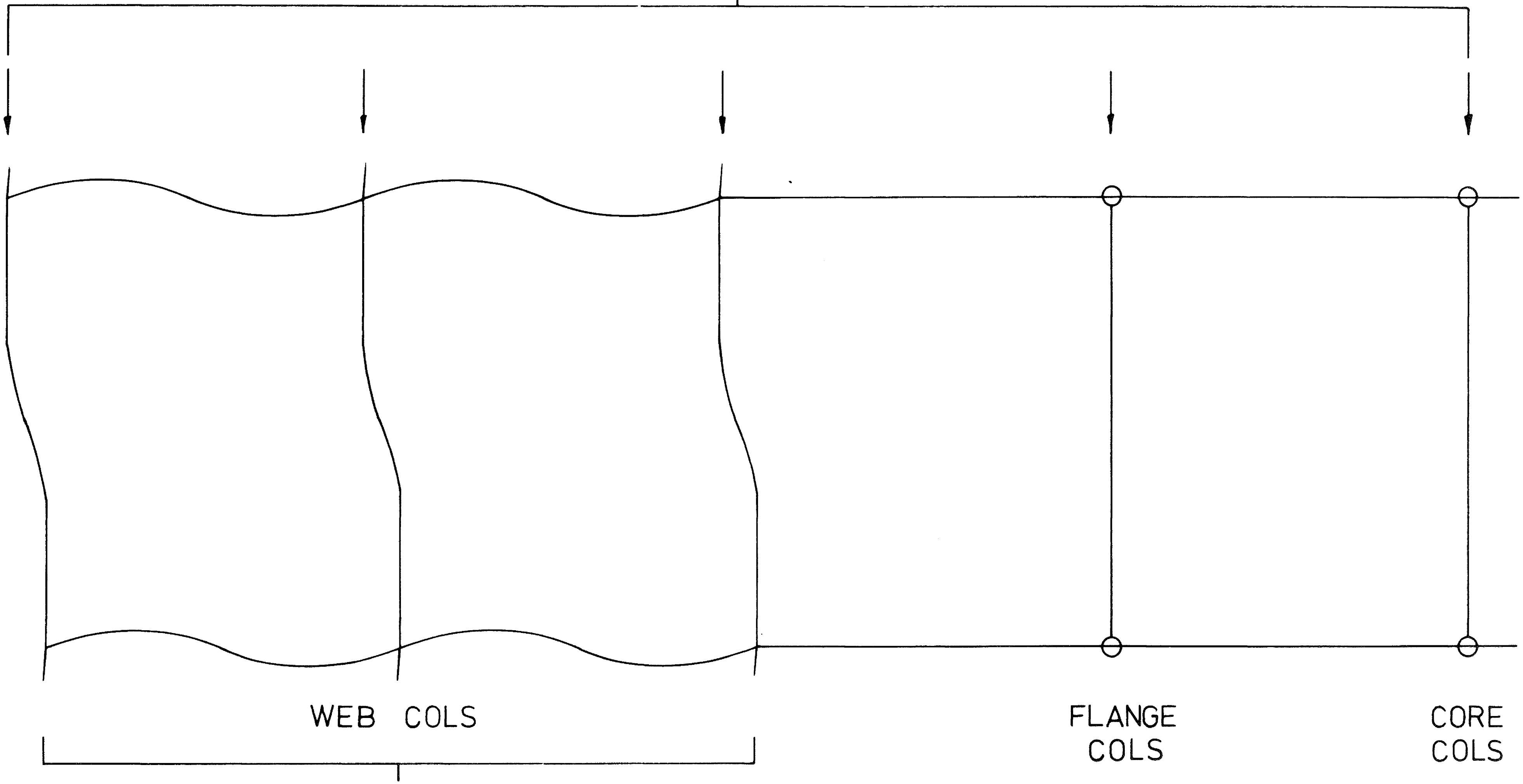
2 EL CENTRO MAY 1940 N-S 5% DAMPING

3 SAN FERNANDO FEB 1971 HOLIDAY INN N-S 5% DAMPING

4 ARTIFICIAL B1 EQ 5% DAMPING

5 ARTIFICIAL A1 EQ 5% DAMPING

FIGURE 6



ΣP_{CR} = BUCKLING CAPACITY OF WEB FRAMES

$$M_{MAG} = \frac{1}{\left(1 - \frac{\Sigma P}{\Sigma P_{CR}}\right)}$$

FIGURE 7: **OVERALL STABILITY CRITERIA**

COLLAPSE MECHANISM

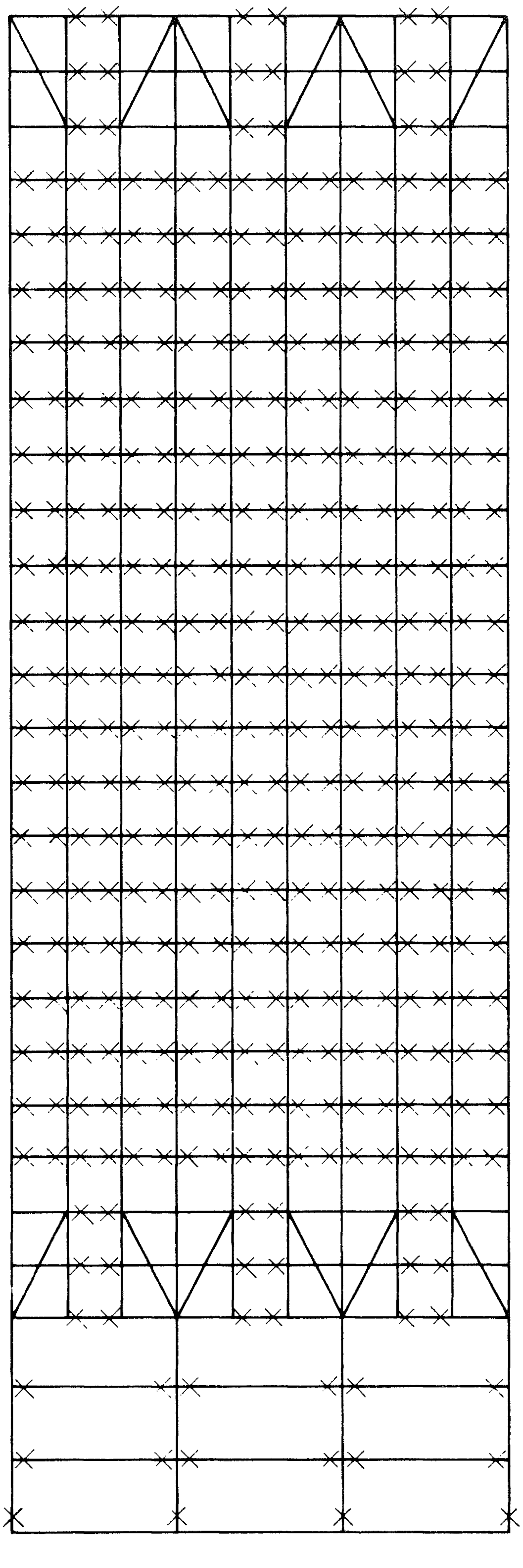


FIGURE 8

FIGURE 9

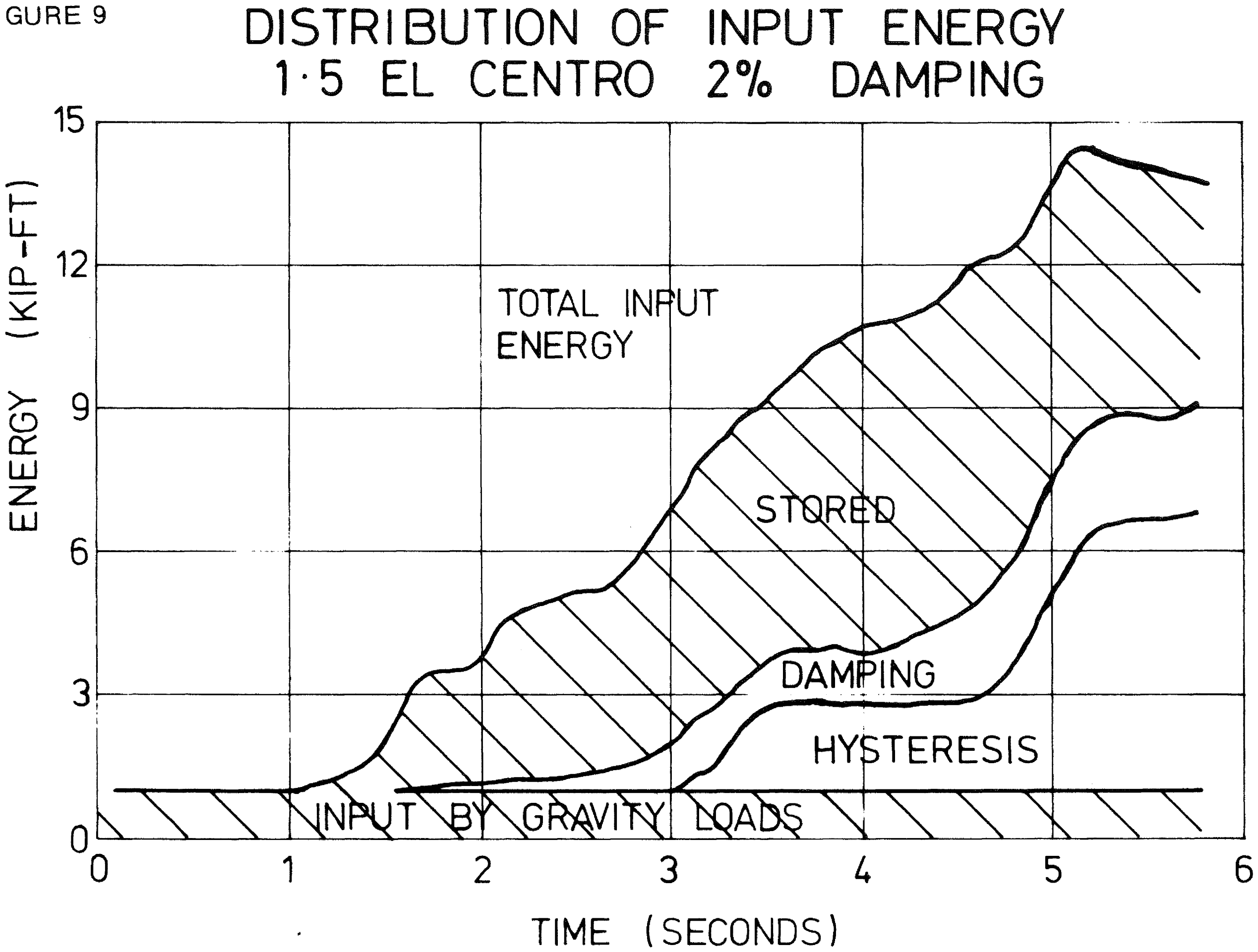
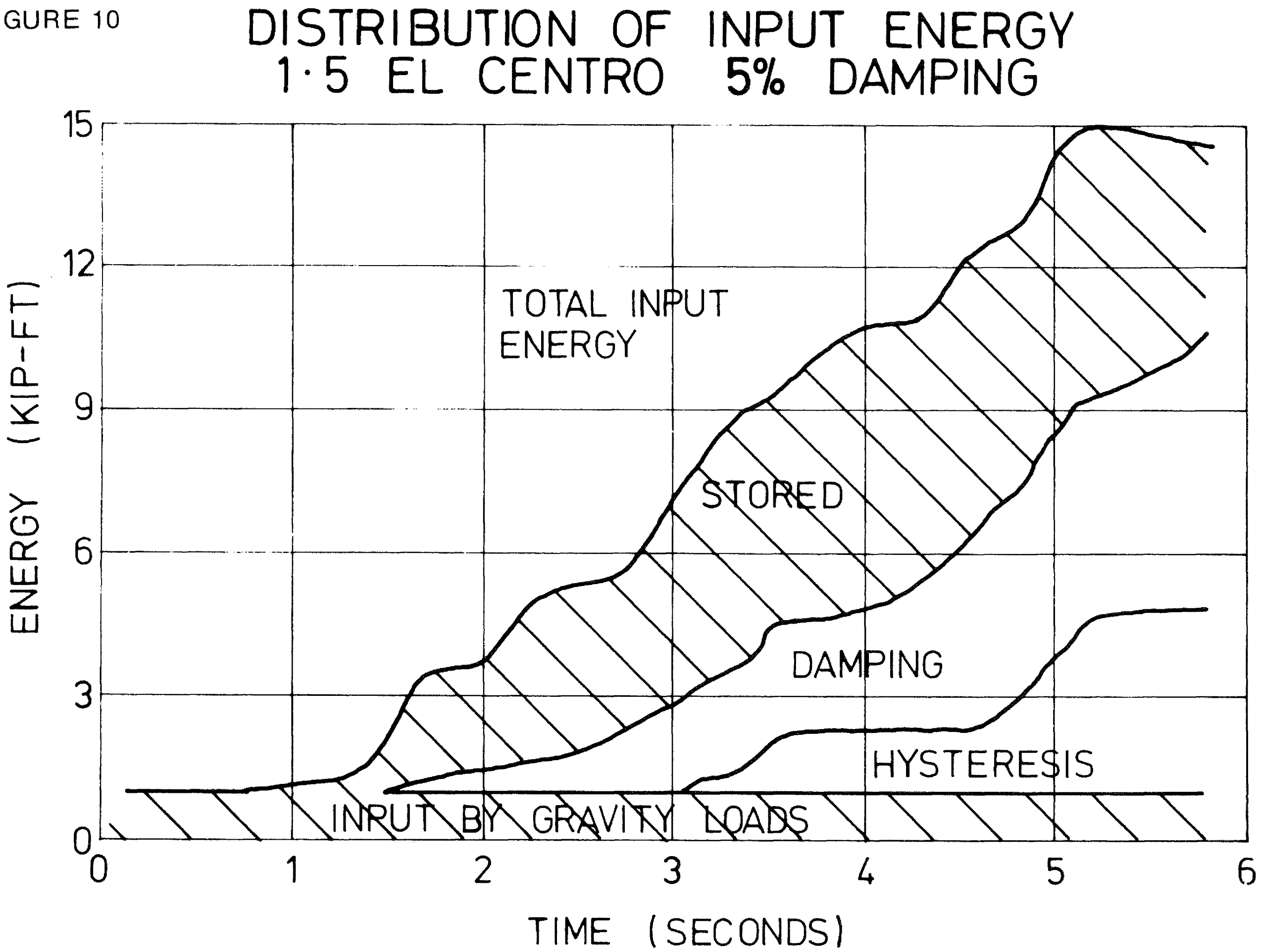
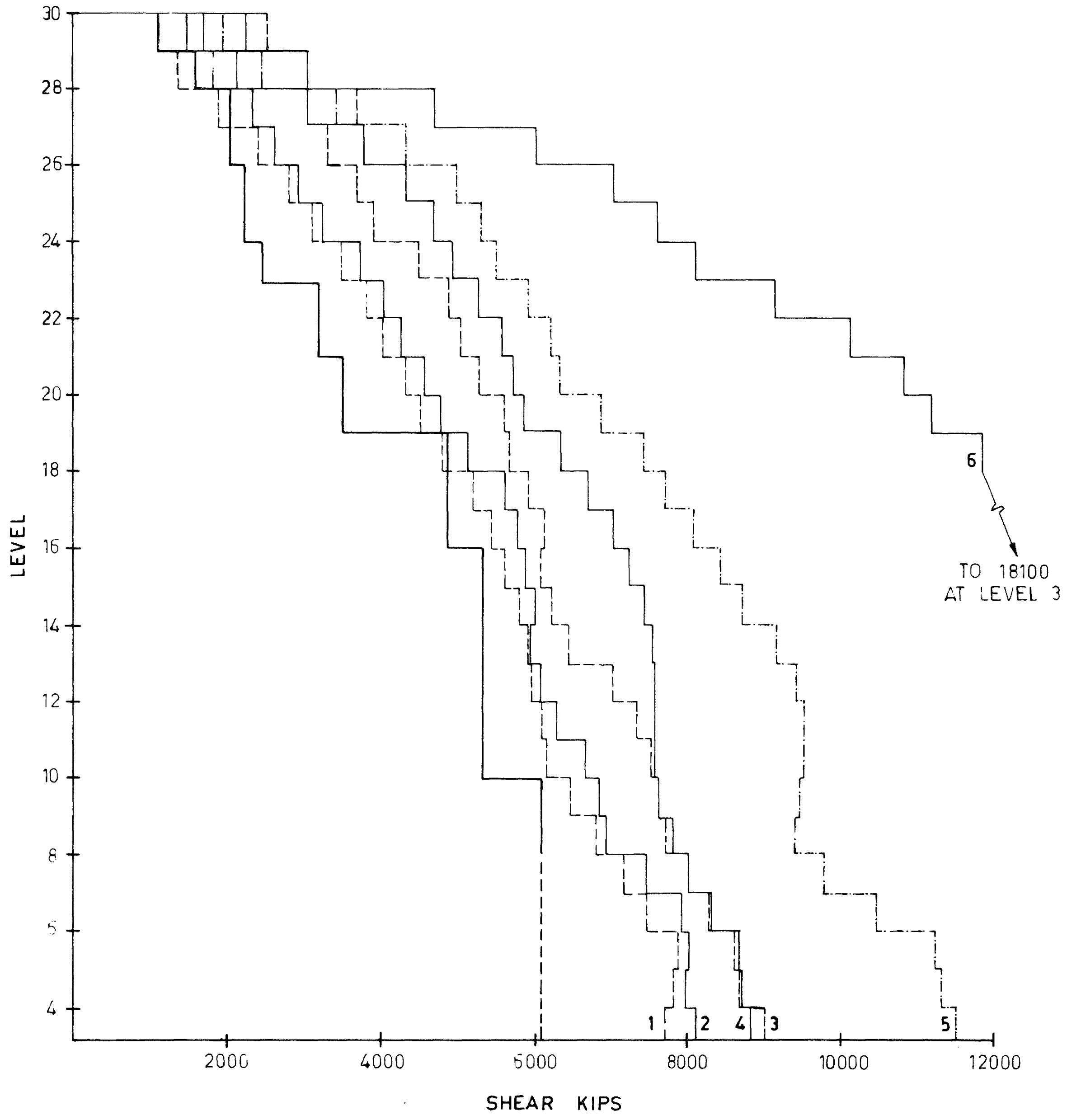


FIGURE 10





1 1·5 EL CENTRO ELASTO-PLASTIC 5% DAMPING

2 1·5 EL CENTRO ELASTO-PLASTIC 2% DAMPING

3 A2 ARTIFICIAL EQ. ELASTO-PLASTIC 2% DAMPING

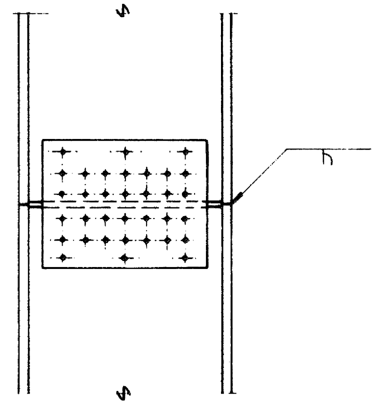
4 1·5 EL CENTRO ELASTIC 5% DAMPING

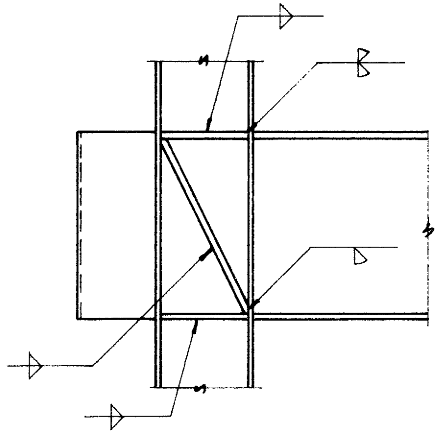
5 1·5 EL CENTRO ELASTIC 2% DAMPING

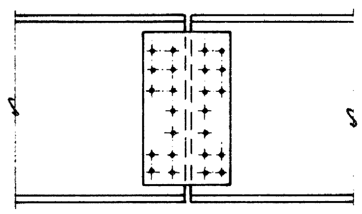
6 A2 ARTIFICIAL EQ. ELASTIC 2% DAMPING

FIGURE 11

TYPICAL CONNECTION DETAILS







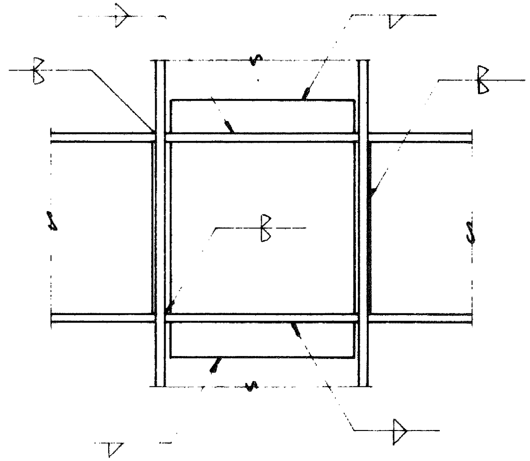


FIGURE 12

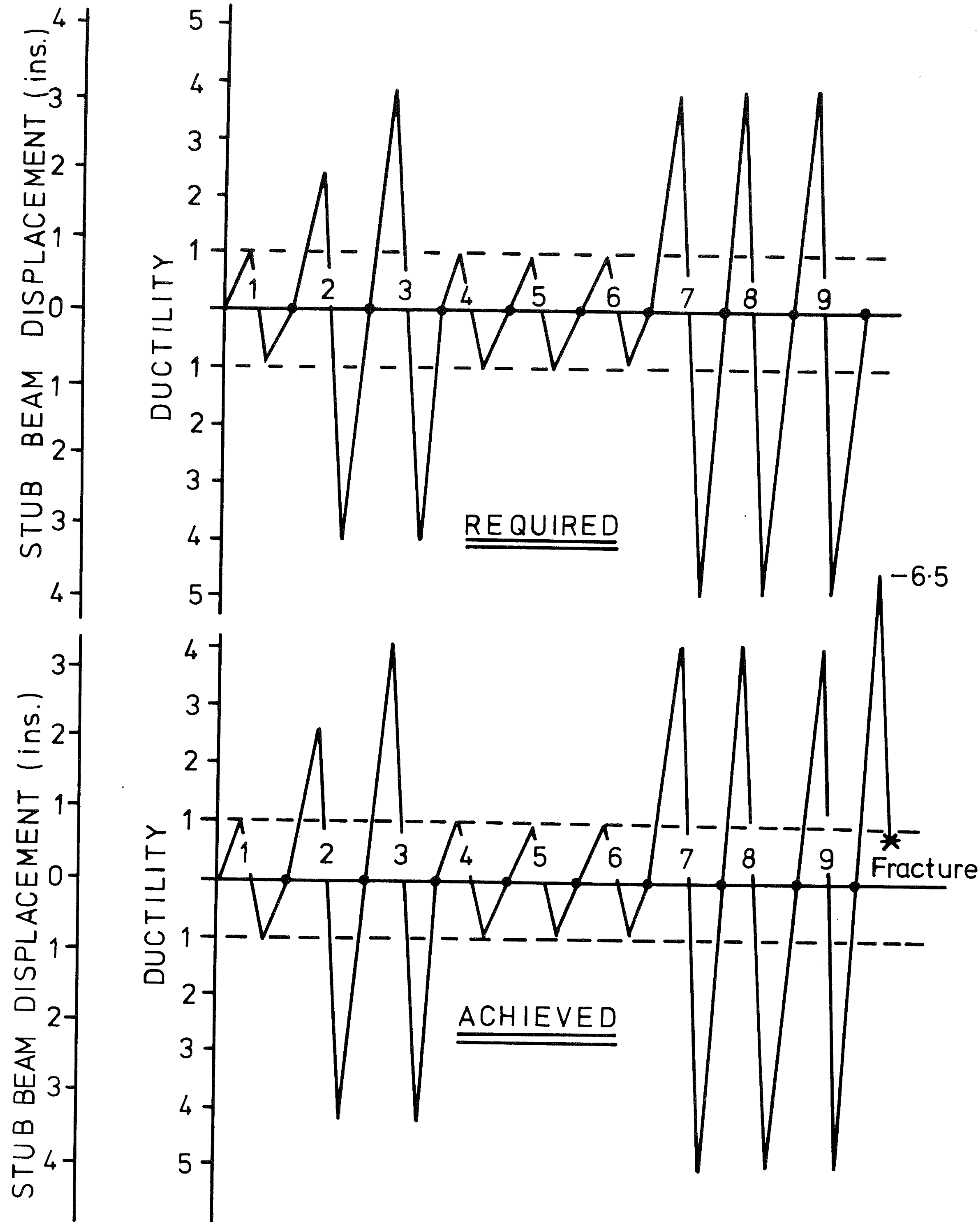


FIGURE 13: LOADING PATTERN, THIRD TEST

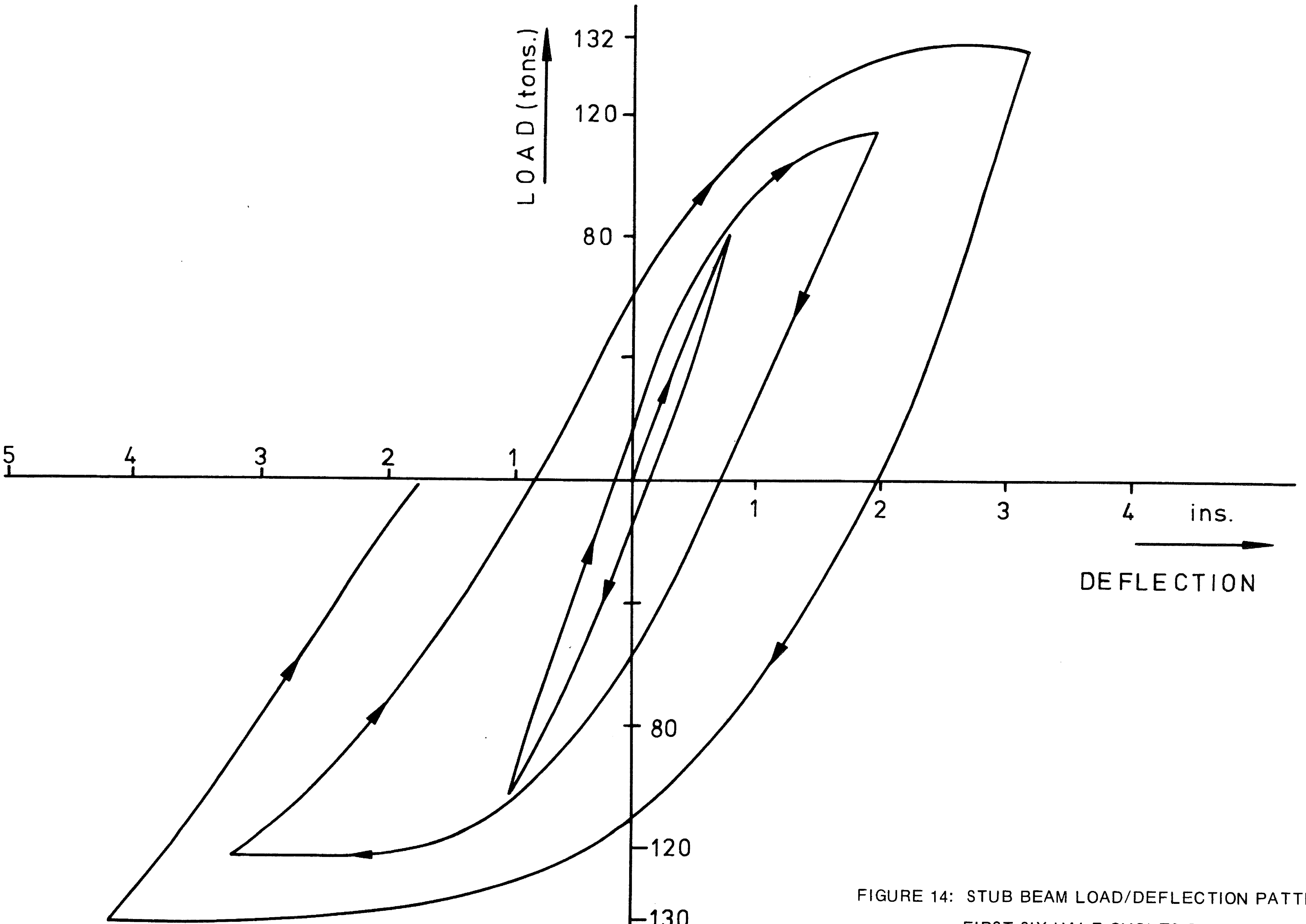


FIGURE 14: STUB BEAM LOAD/DEFLECTION PATTERN,
FIRST SIX HALF CYCLES OF LOADING

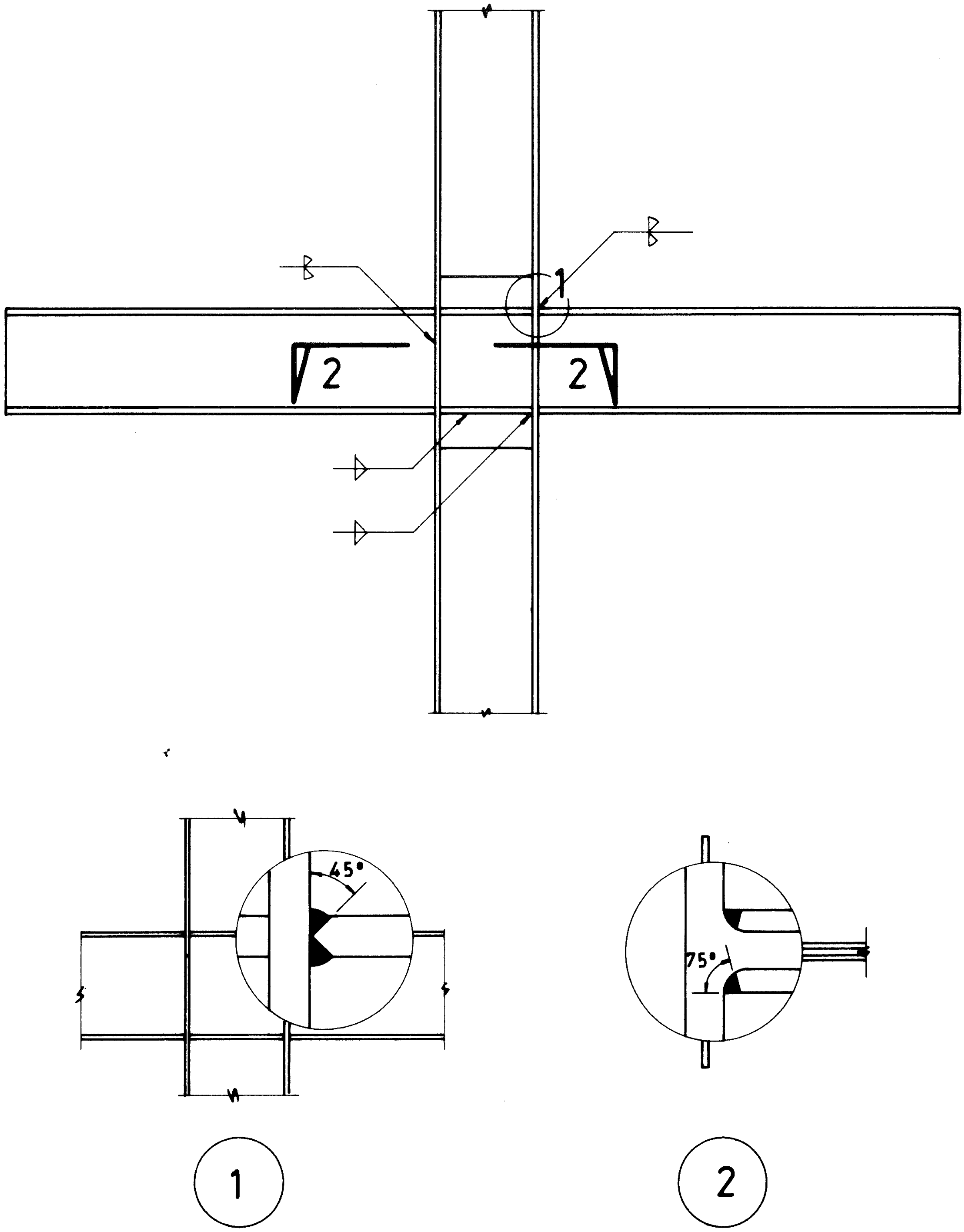


FIGURE 15: TEST JOINT

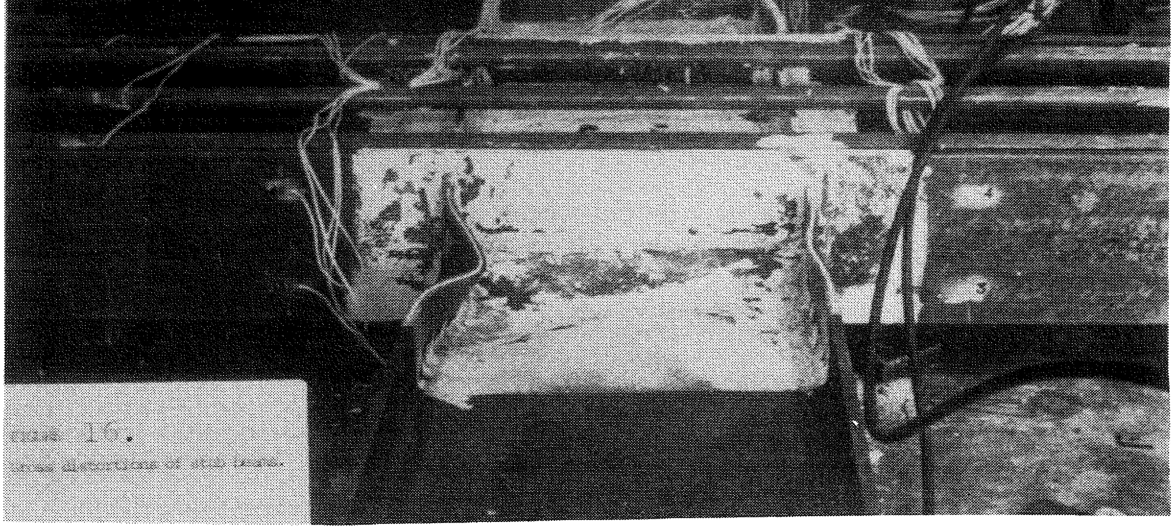


FIGURE 16.
Gross distortions of stub beam.

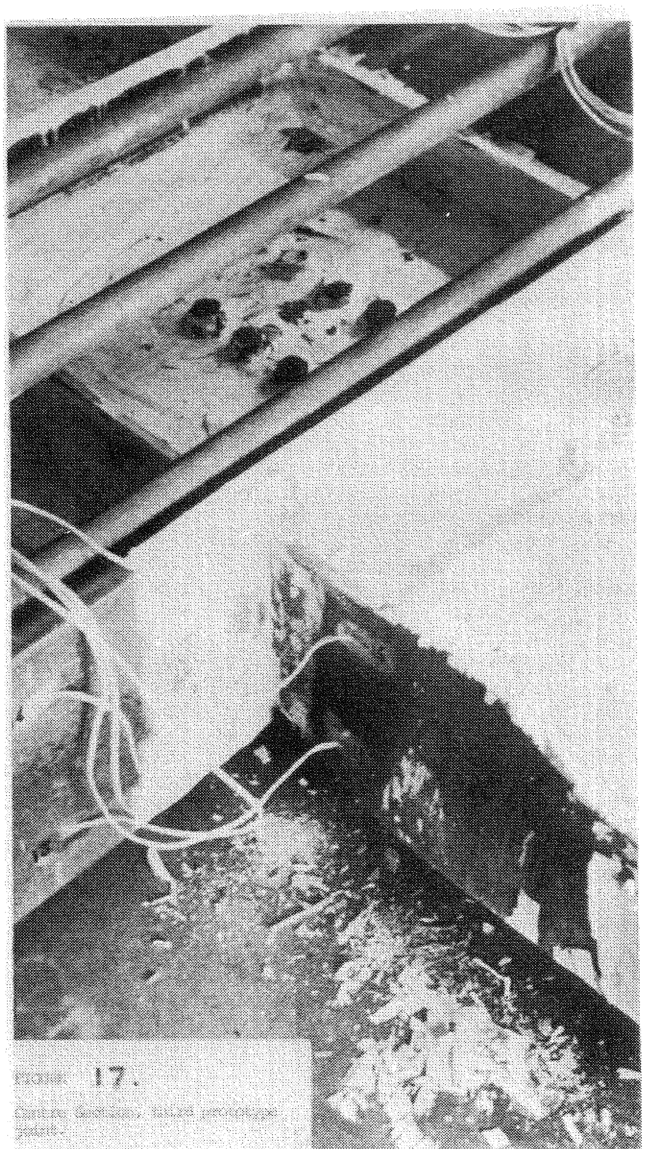
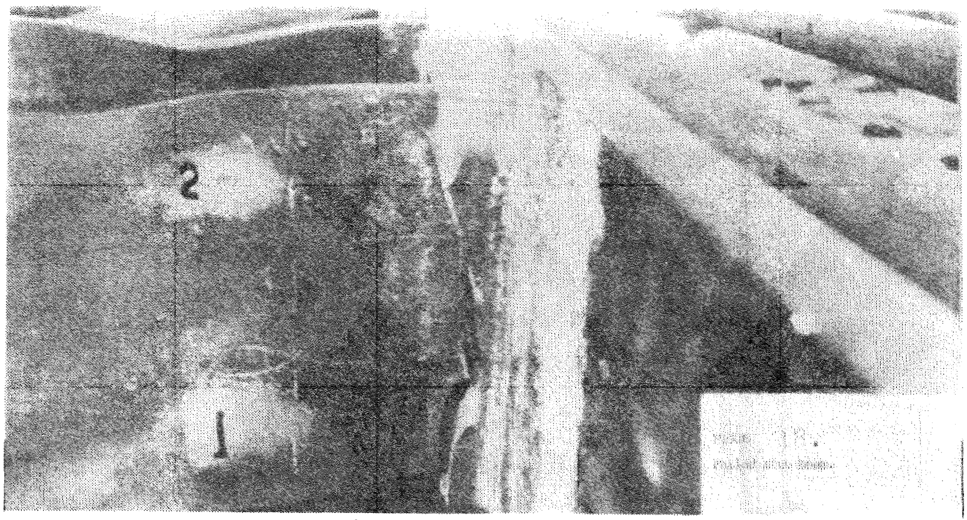
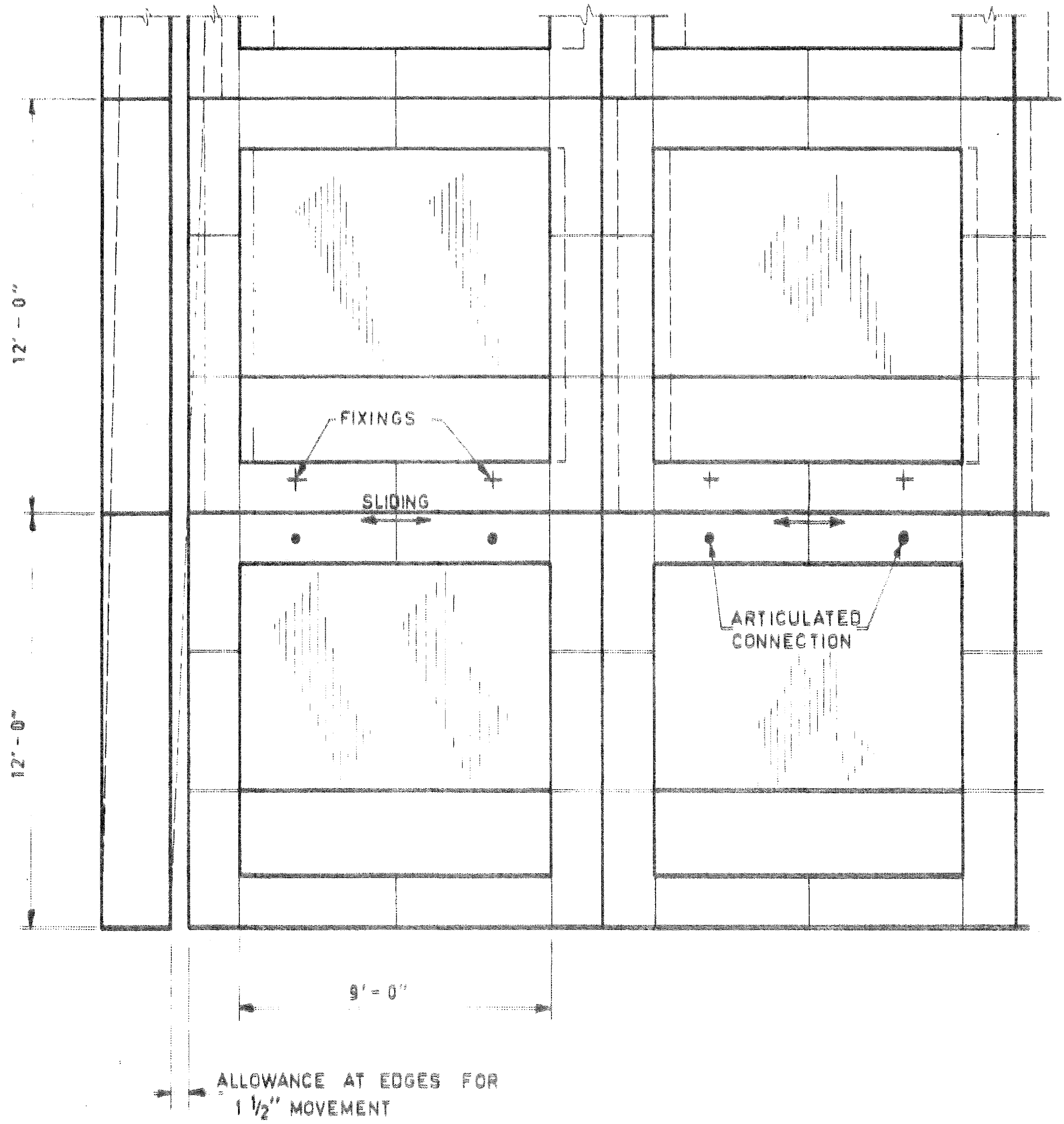


FIGURE 17.
Centre Section, third prototype joint.





FACADE PANELS

FIGURE 19

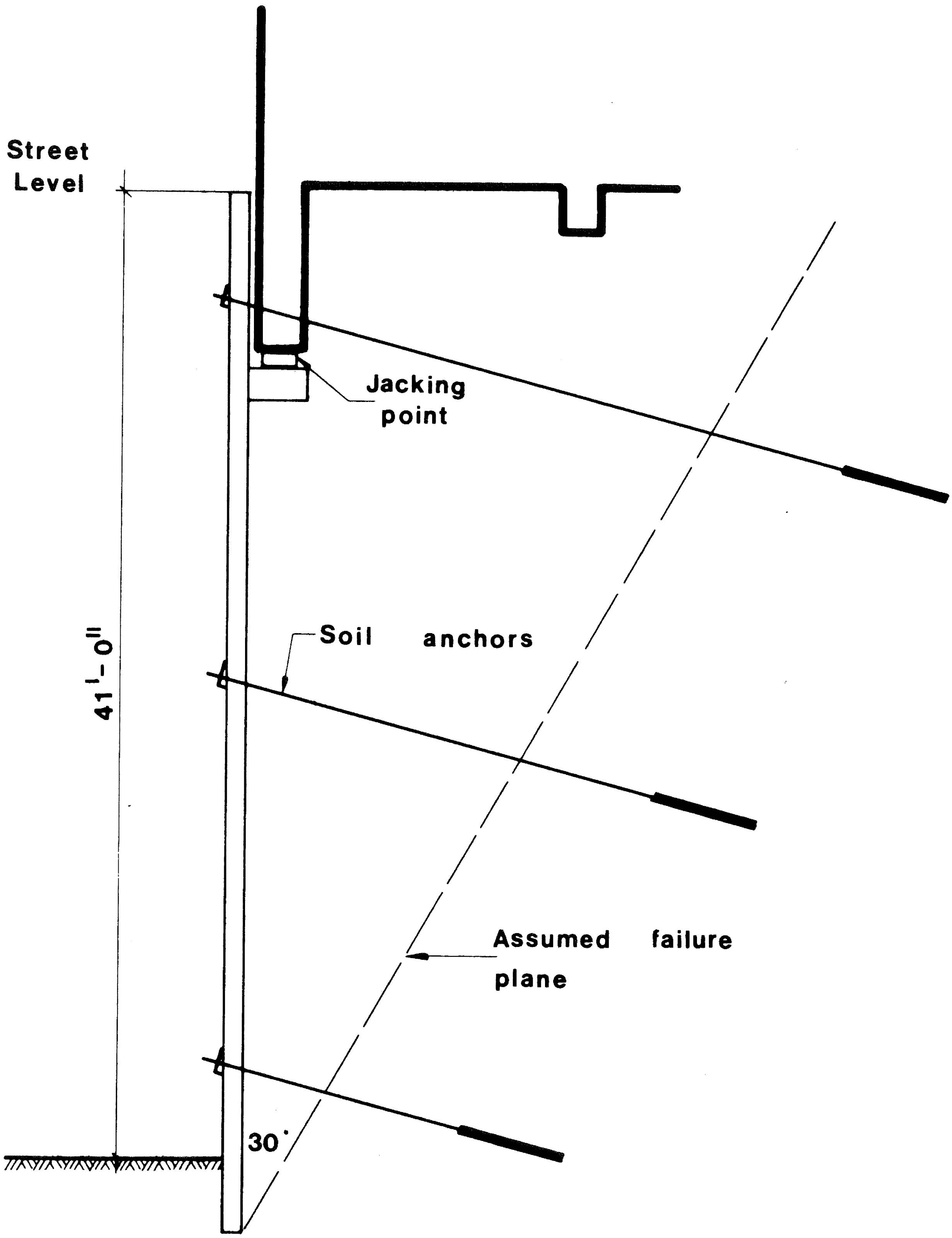


FIGURE 20: SHORING SYSTEM