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Seismic resistance of a moment resisting steel portal frame with composite concrete slab deck and simulated gravity loading

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ABSTRACT

The seismic resistance of structural steel moment resisting frames with composite concrete floor slabs which are subjected to appreciable gravity loading has received little attention from researchers. The gravity load can cause unidirectional plastic hinges to form in the beam with a high rotational ductility demand.

The majority of experimental tests reported to date for frame systems have concentrated on statically determinate sub-assemblages of beam/column joints. Relatively little testing has focussed on indeterminate test specimens which have more complex behaviour characteristics. This bias is reflected in code documents, for example, which discuss performance goals in terms of member ductility capabilities. Apparently obvious definitions such as "first yield" need to be reviewed when applied to indeterminate systems.

Reported in this paper is the observed response of a portal frame which was subjected to simultaneous vertical (gravity) and lateral loading. The frame comprised steel universal beam sections with a poured slab (115mm) atop a proprietary steel tray deck. The dimensions were selected to approximate at 3/4 scale a realistic floor system. Composite action was achieved with standard welded steel studs.

Introduction

Prior to 1989, composite slab construction was not addressed in the New Zealand steel design code. In 1989 NZS 3404:1989 was published and for the first time detailed provisions were included for composite construction. To date little experimental research has been undertaken within New Zealand to investigate the behaviour of composite beam structures. Consequently when drafting NZS 3404:1989 extensive reliance was placed on the results of overseas studies. The new New Zealand rules draw heavily on Canadian design practice in particular, since Canada has adopted limit states design methods which are consistent, at least in the non-seismic part, with New Zealand design philosophies (refer CAN/CSA-S16.1-M89, 1989, for example).

NZS 3404:1989 contains some limited guidance for special detailing of the composite member for ductile action. The provisions discuss the positive or negative moment region at a support and the positive moment region within the span. In the region near the support the more simple (and predictable) solution is to curtail the composite action at some distance away from the support

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face and design based on the capacity of the steel beam alone. For the midspan region there is a suggested rule based on the plastic strain in the steel section compared to the ultimate compressive strain in the slab (for a category 1 member the Code requires that the steel beam shall achieve a maximum tensile strain of 24 times yield strain prior to the ultimate compressive strength of the concrete slab being attained and a maximum concrete strain of 0.003 being exceeded). It is not common to expect a midspan hinge as a result of seismic loading but the possibility cannot be ignored in frames which have high gravity loads.

The two goals of the test program reported here were therefore to design a steel portal frame with a composite deck slab and to study (i) the region of the beam adjacent to the beam/column joint and (ii) the midspan region. For reasons discussed later in this report there was limited success in forming and maintaining the in-span hinge.

Experimental Program

Portal Geometry

Figure 1 shows in elevation one half of the span geometry (apart from the loading details - the lateral load was applied as either tensile or compressive at one end only, and the central loads were slightly eccentric from the beam span centreline - the portal was symmetrical about its mid-span centreline).

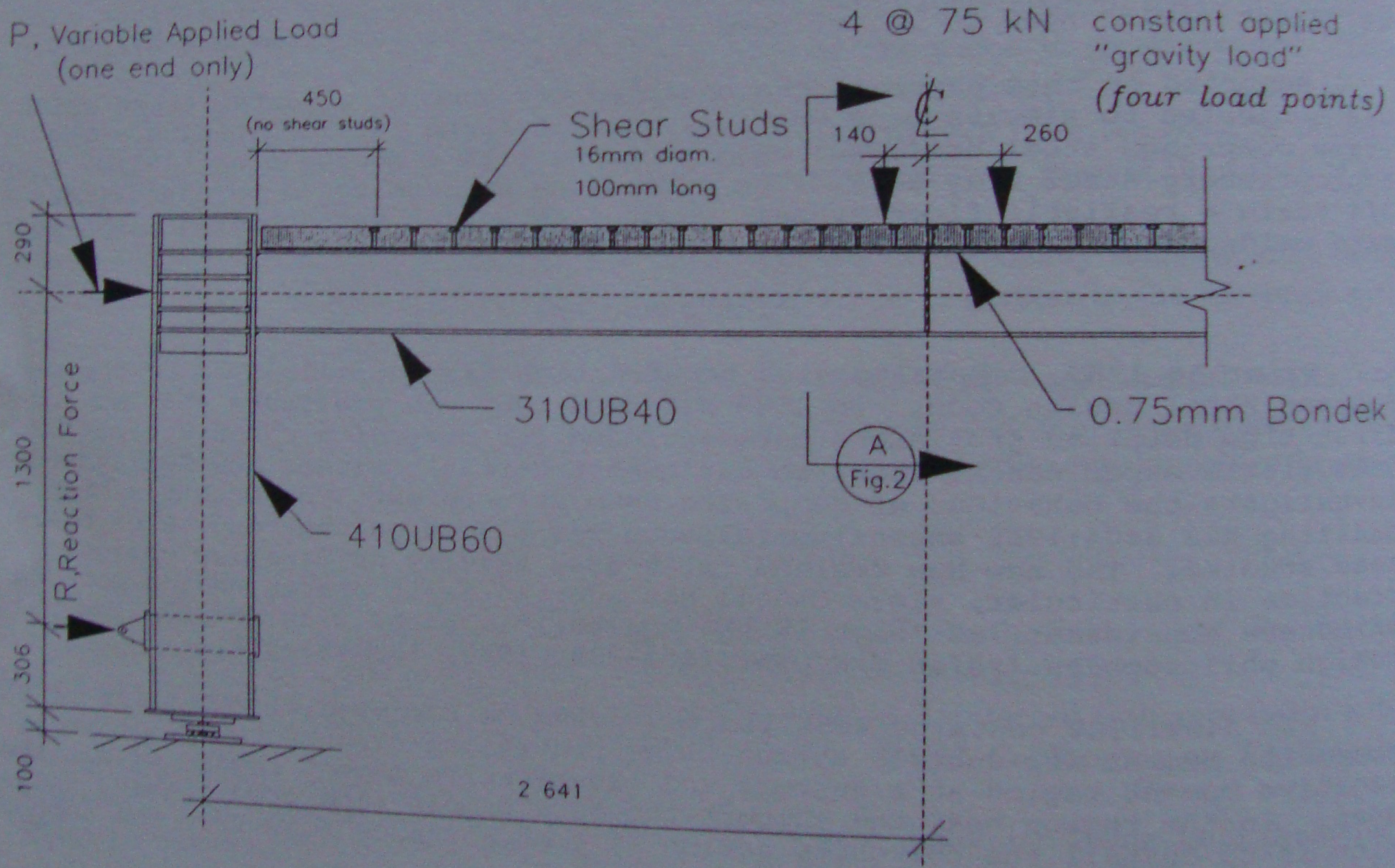


Figure 1. Elevation of Portal Frame

The two inner pairs of web stiffeners were eliminated in the beam-column joint remote from the horizontal load application end. The portal was detailed as pinned base. To determine the bending moments in each portal leg during the testing, the horizontal reaction was measured by means of a load cell near each column base. For this reason the columns were restrained by a horizontal link approximately 300mm above their base, which provided translation restraint but rotation release. To maintain the simple pinned support it was necessary to detail a roller bearing detail under the column baseplates.

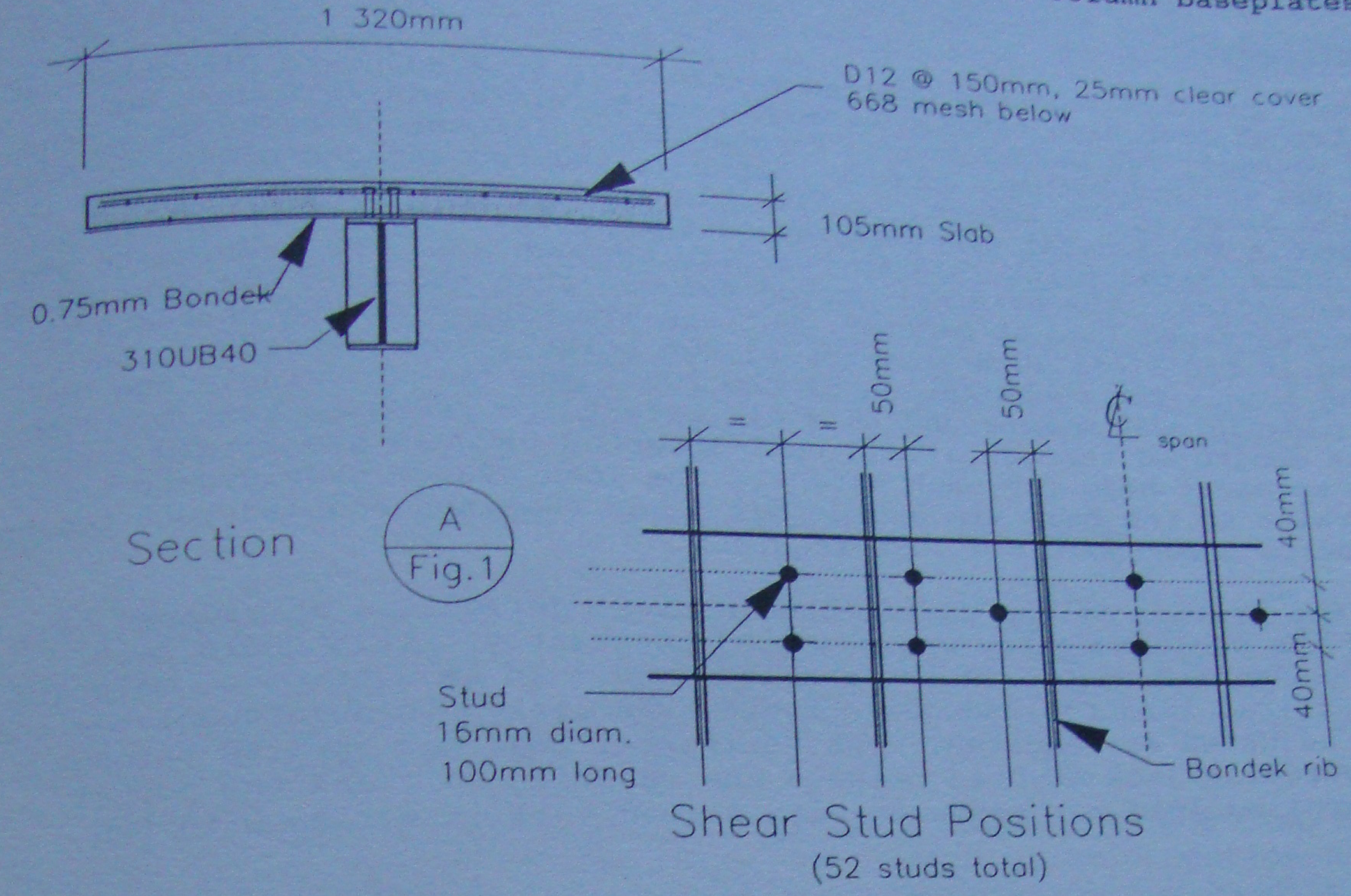


Figure 2. Section Details of the Full Composite Beam and Slab

The typical section details are given in Fig. 2. The shear studs were 16mm diameter and 100mm long and were welded through the deck pan to the top flange of the Universal Beam section. Composite action was curtailed at the ends of the beam span by eliminating any studs within 450mm (1.5 x beam depth) from the column face.

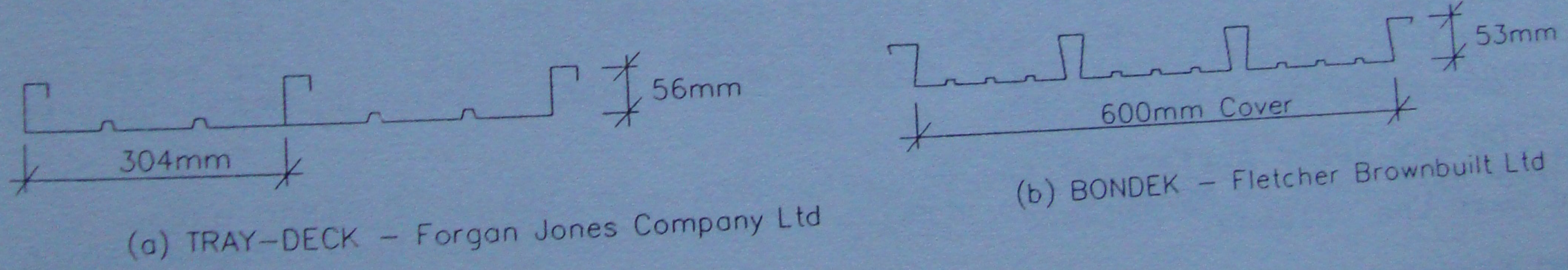


Figure 3. Comparison of Steel Deck Profiles

The original design was based on the TRAYDEK profile which produces a solid composite slab. The fabricator chose to substitute BONDEK on the assumption that it was an equivalent product. BONDEK produces a hollow composite slab, however, and a simple substitution was not adequate. As a result of this substitution the original 75mm long studs installed were too short and did not satisfy the Code minimum requirements. The studs were ground off and the beams refabricated with 100mm long studs. The depth of the slab had to be increased from 80mm to 105mm to permit full embedment of the longer studs. This increase in slab depth resulted in an increased midspan moment capacity without a corresponding in the capacity near the support (since there was no composite action in this region). As discussed later this modified the intended behaviour of the portal during the test.

Standard concrete cylinder tests established a concrete compressive strength of 29MPa. Coupons tests from steel samples determined a steel yield strength of 310MPa and an ultimate strength of 465MPa.

Loading Arrangement

The fixed "gravity" load was applied vertically, in a downward direction, at midspan through four high tensile strands coupled via a manifold to a common hydraulic source. The cables passed through 50mm diameter holes bored in the deck slab and spanned between spreader beams at the top and the underside of the reaction floor at the bottom. A flexible loading system was necessary in order to permit the lateral displacement of the portal. Each spreader beam applied a concentrated point load to the slab through a rocker bearing at the beam web centreline. The original design assumed that the load was applied as a single concentrated load at midspan, but the limitation of the strand capacity resulted in the final load geometry of two point loads instead of the single point load. The substitution of the two loads for the single one did reduce the midspan peak moment and this did affect the proposed test program. The load was monitored through a load cell inserted between the strand anchorage and the spreader beam for each strand. The load system provided no torsional restraint to the beam since the four jacks remained coupled to a common hydraulic source throughout the test sequence.

During the cycling of the portal the vertical deflection at midspan of the beam and the cable geometry was changing constantly. This could have caused a consequential change in the applied load. In an attempt to maintain a constant vertical load the hydraulic system was linked to an accumulator and the pressure adjusted as required. The horizontal load was applied through a horizontal hydraulic jack mounted between the reaction wall and the portal frame. The applied load was measured with a load cell in series with the jack.

Instrumentation

One side of the beam was instrumented with portal-type displacement gauges set in a cross-braced truss pattern. The top and bottom chords of the truss measured the extension/compression of the bottom and top flanges of the beam section. Addition portal transducers were mounted at each end of the beam to measure total elongation/shortening of the beam and to measure deformations within the joint region. Portal displacement gauges spanned between the top flange of the steel beam and the slab soffit to measure any slip at the interface. A line of twelve portals down the centreline of the top of the concrete slab recorded strains in the slab. The gauges in this array were scanned semi-automatically via a Phillips Multichannel Scanning Datalogger to Personal Computer file storage.

On the upper surface of the concrete deck a grillage of DEMAC gauge points were used to measure the strains in the concrete in the midspan region and near the columns.

Load Sequence

Before the lateral load test, the portal was subjected to several loading and unloading cycles of vertical load only to verify the linearity of the system under elastic load levels.

The subsequent load tests were displacement rather than load controlled. A typical sequence beginning with cycles to 75% of "yield", then "first yield", and subsequent cycles to displacement ductilities of 2, 4 and 6 was envisaged. A definition problem arose however. Most tests undertaken by others in the past, and which have established the cycles of loading behind Code ductility rules, have been undertaken on statically determinate subassemblages. The important point being that there is no distinction between first "yield" ("yield" in this context implies full plastic moment at a section rather than first yield of the extreme outside fibre) and the formation of a collapse mechanism for such systems. There is a difference between first yield (i.e. the first hinge) and the formation of sufficient hinges to form a collapse mechanism for an indeterminate frame (such as the pinned based portal selected for this study).

The strategy adopted for this project was to first calculate the horizontal load intensity necessary to form the plastic collapse mechanism for the portal (250kN). This value was calculated theoretically, using the best estimates available for the material properties. The initial elastic lateral load-lateral deflection curve was plotted and the straight line curve extrapolated to find the displacement required to reach the theoretical collapse load (15mm). This can be considered a conservative displacement prediction since the portal stiffness was expected to soften with increasing load - particularly in the interval between the formation of the first hinge and the formation of the collapse mechanism. The displacement thus defined was treated as the "yield displacement" and subsequent cycles took the frame to 2, 4, and 6 times this displacement.

Theoretical Prediction of Beam behaviour

Prior to the load test, the anticipated performance of the portal frame was determined from plastic analyses. The results of the theoretical analyses are summarised in Steps 1-4 in Fig. 4. The horizontal load required to form the collapse mechanism was 250kN and the corresponding horizontal displacement 21.7mm. The equivalent figures at formation of the first hinge are a load of 92kN and a displacement of 3.8mm.

The flexural strength of the composite section of the beam member, 371kN-m, in the portal was based on the simple plastic section analysis recommended in NZS3404:1989. This value was supported by the results of a vertical load test of a simple span composite beam reported separately (HERA Report, 1990).

Figure 4 follows the predicted response of the portal frame through several cycles of lateral loading and contains several points of interest. Step 1 gives the bending moments which result from the self-weight of the portal and from the vertical "gravity load". During the first cycle of loading a hinge forms first at the beam face of the column remote form the applied lateral load (Step 2). During this increment of load the sway moments have little effect on the gravity moments at midspan. Beyond Step 2, however, the moments increase in proportion to the distribution identified as "Sway 2" in Step 2a. It is this step which introduces the possibility of a mid-span hinge. According to the idealised model, the second hinge (and mechanism) forms near the column face at the point of termination of the shear connectors, Step 3. (The mathematical model assumed a step change in member plastic moment capacity at this point.) The moments at midspan are approaching 95% of the section capacity.

Steps 3a & 4 trace the reversal of lateral load back to zero load. There is a substantial redistribution of the gravity moments in this new equilibrium position compared to the initial distribution in Step 1. Steps 5 through 7 trace subsequent cycles and demonstrate that little further redistribution of gravity moments is expected after the first mechanism forms. The lateral load required to form the first hinge increases fourfold as a consequence of the "gravity" moment redistribution, however the load required to form the collapse mechanism is little changed.

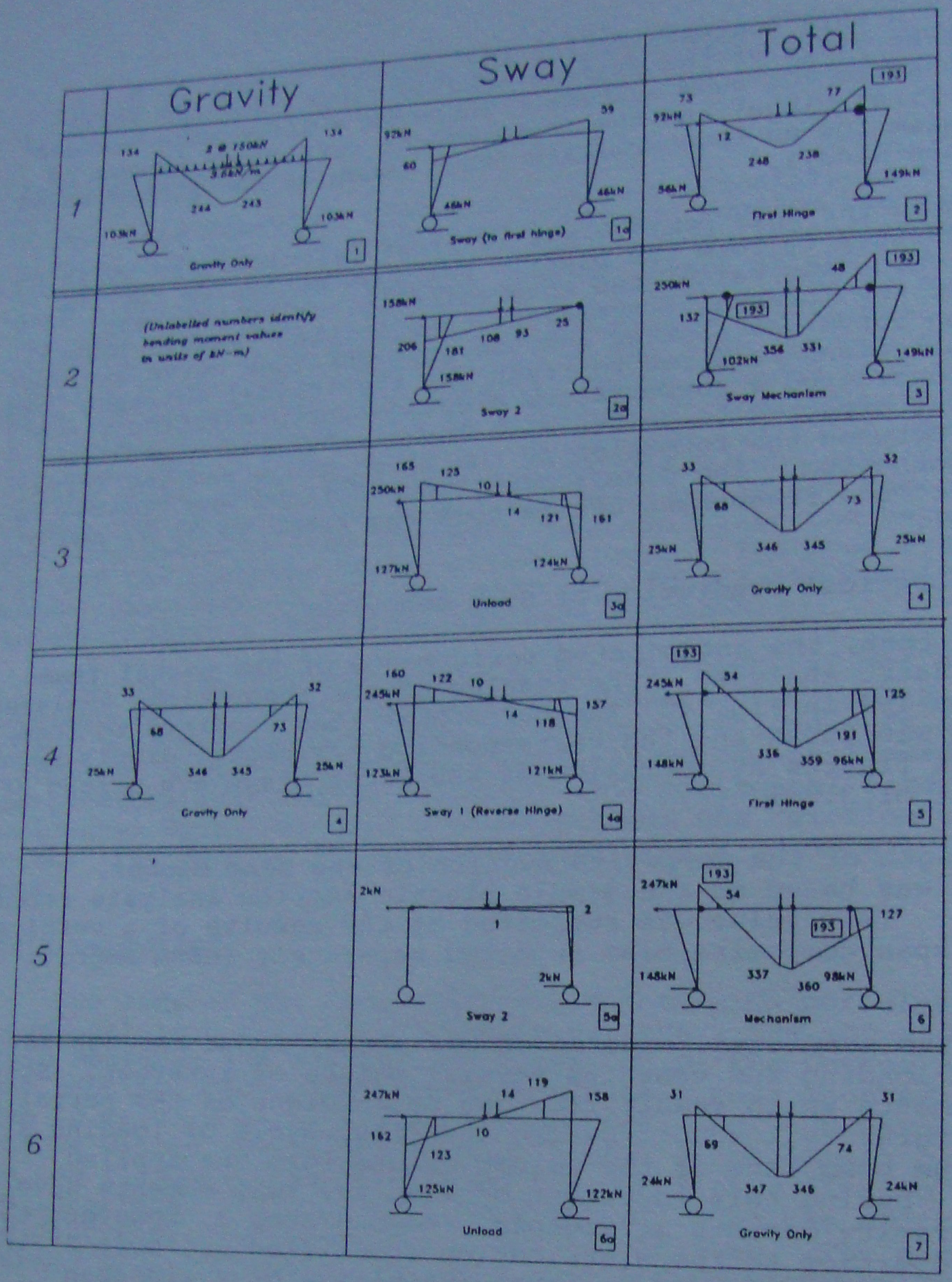
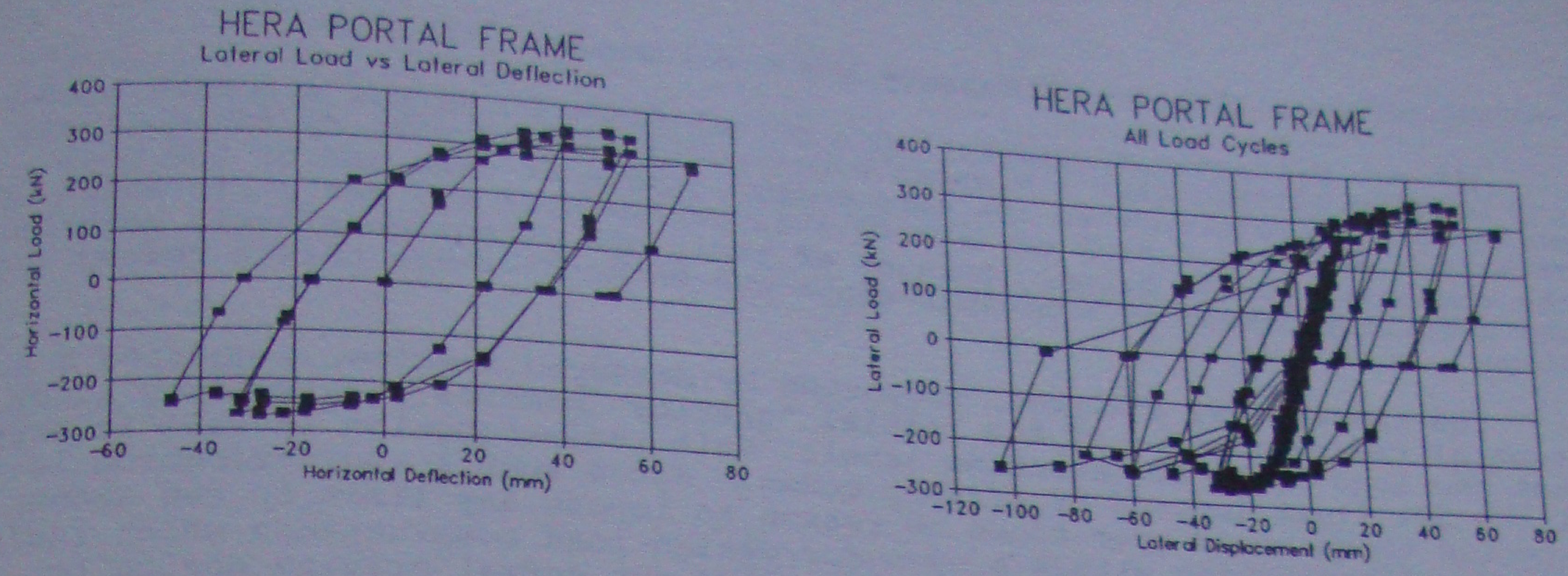


Figure 4. History of Sway Mechanisms and Moment Redistribution

Observed Response

In Figure 5 the cyclic variation of lateral load versus portal lateral deflection is plotted. The oval shape of the resulting plots suggests a robust seismic performance. Up to the end of the "ductility 4" cycle there was little strength loss, but beyond this level there was a softening of the structure. Large cracks began to open in the deck surface, radiating out from the shear stud locations in the deck at this displacement level.

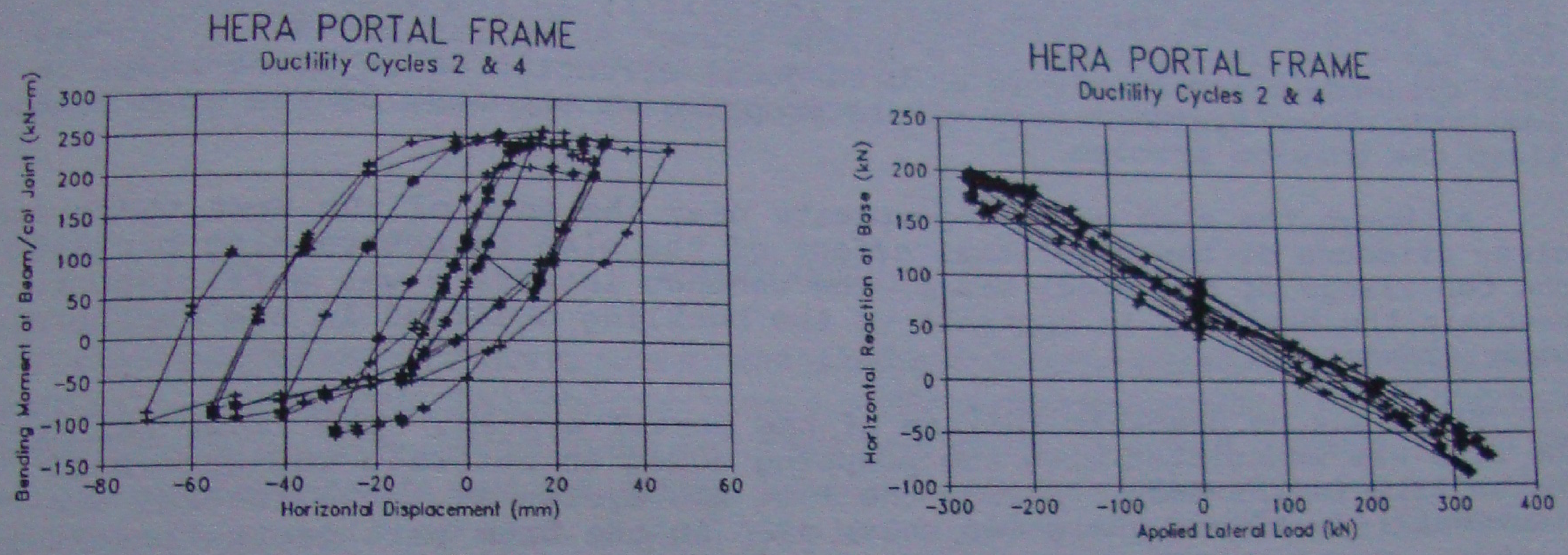
Of interest for this paper was the collapse mechanism formed by the structure, in particular evidence of substantial gravity moment redistribution as predicted by the theoretical model.



(a) Ductility cycles 2 & 4

(b) Composite of all Cycles

Figure 5. Summary of Applied Horizontal Load versus Measured Horizontal Deflection during Testing Cycles



(a) Beam/Column Joint Moment vs Applied Lateral Displacement

(b) Column Horizontal Reaction vs Applied Lateral Load

Figure 6. Summary of Bending Moment, Applied Lateral Load and Applied Lateral Displacement during Ductility 2 & 4 Testing Cycles

The redistribution of gravity bending moment is confirmed by the change in horizontal reaction as measured at the base of the portal (the portal column remote from the applied lateral load is plotted) which is plotted in Figure 6(b). The initial reaction, under gravity only, of approximately 100kN relaxes to less than half this value during successive cycles. The peak bending moment at the joint (Figure 6(a)) approaches 275kN-m. This value exceeds predicted value of Mp=193kN-m because of strain hardening effects and because the plotted value is at the joint centreline whereas yielding takes place at the column face.

Summary and Conclusions

It was difficult to proportion a portal frame to have a strength hierarchy which predicted that a midspan hinge would form as part of the collapse mechanism in preference to the more common hinges at each end of the beam. This is a point in favour of, rather than detrimental to, the performance of the composite portal frame. Because of their one direction plastic rotations the ductility demand on a midspan hinge can be more severe than a column face hinge.

The theoretical analyses, supported by measured results, suggested a substantial redistribution of the initial "gravity" moments particularly after just one load cycle to mechanism level. This redistribution is well in excess of the 30% limit imposed by current codes. Since such a redistribution is expected there would seem to be no reason to impose any limit on the amount of moment redistribution within a span for design load combinations which include seismic loads. The limit should be retained for normal gravity design to ensure a minimum strength distribution and acceptable service state performance.

The portal demonstrated a very stable resistance to the applied lateral loads with little pinching in the horizontal load vs lateral deflection hystertic loops. There was some torsion instability in the deck of the portal but this was a result of modelling a "plane frame" slice rather than a two-way deck spanning concurrently in an orthogonal direction to adjacent beams in a realistic floor system. Some simple propping at the ends of the span stabilised the torsion problem.

Although the slab was non-composite near the ends of the span there was clear evidence of the beneficial effect of the slab in restraining buckling in the top flange of the steel beam. The contact interface was sufficient to restrain the buckling in contrast to the buckling observed in the unrestrained lower flange.

There was no dramatic failure of the portal system. The final failure of the slab was associated with the propping added to restrain the torsional instability in the deck. During the last few cycles at large lateral displacement (+/- 100mm) there was shear slip in the beam part of the beam/column joint.

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