A STEEL BEAM CONCRETE COLUMN FRAME SYSTEM

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SUMMARY

In this paper a plane frame system designed to resist earthquake induced loading is described. The frame consists of reinforced concrete columns and rectangular section structural steel beams. The planning of the framing system from both engineering and architectural view points, along with the limitations the system imposes on these disciplines, are considered. Design problems are identified and their resolution is described. Finally the construction aspects of workmanship, speed and cost are reviewed.

INTRODUCTION

In order to obtain the ductility benefits of structural steel beams in seismic resistant frame design, but still avoiding moment resisting beam-column connections in structural steel which entail either site bolting or welding, a framing system has been developed using structural steel beams and reinforced concrete columns. An added advantage of maintaining reinforced concrete for columns is that additional fire protection of the columns is not required and the natural concrete finish can be used to architectural advantage. The steel beams which are rectangular in section pass through the concrete columns and thus transfer the shear and flexural forces from the beam to the column by bearing. The system is ideally suited to lateral load resisting elements situated on the external faces of the building. The columns need to be relatively close spaced with the dimension in the plane of the frame of the same order as the clear column spacing. Columns of approximately 1000 x 300mm cross section with 1000mm clear spacing have been used. Limiting the seismic frame to the exterior of the building enables interfloor heights to be reduced by using flat slab construction while still maintaining an acceptable window head height as the steel beams are of limited depth, typically 300mm in the bottom floors of a 14 storey frame.

THE SYSTEM

The Frame

A typical frame which has been incorporated in a building recently completed is shown in figure 1a with a detail of one of the beams relative to the floor slab and columns detailed in figure 1b.

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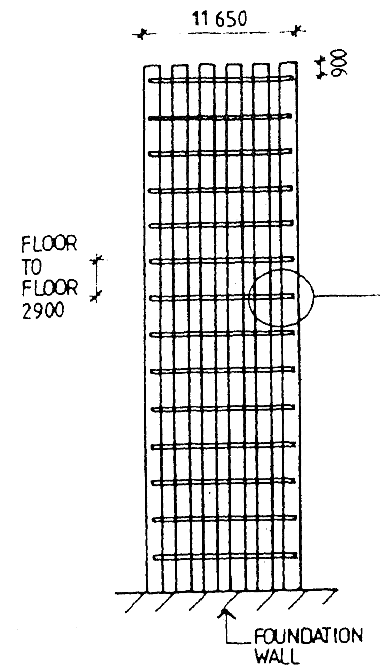

Figure 1a

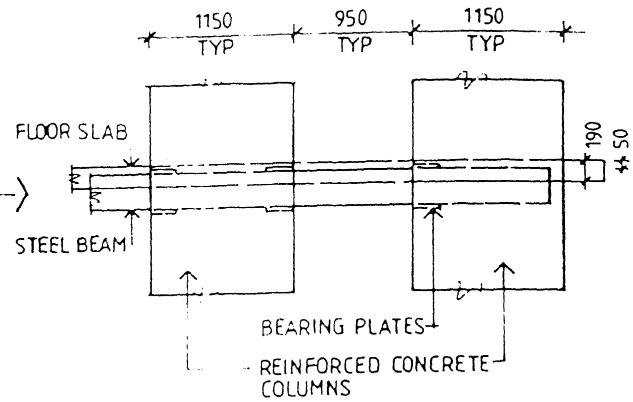

Figure 1b

Elevation of typical frame

The columns range in size from 1150 x 250mm for frame internal columns to 1150 x 400mm for the end column in each frame. The larger size of the end column is necessary so that sufficient reinforcement is available to resist the high tensile forces induced under lateral loadings even though there are no concurrency effects. Because of the close column spacing there is little axial gravity load to suppress tension forces in the end columns.

In this frame the beams range in size from 150 x 40mm at the top of the building to 300 x 50mm at the bottom. The beams and bearing plates have a nominal yield of 250 MPa and were stripped from plate. It is necessary to have true edges which are not readily available with rolled sections. The stripped rectangular sections required only light grinding to produce flat square edges. The bearing plates range up to 244 x 218 x 40mm thick for the largest beams.

Typical Use

The framing system to be described has been used in four completed multistorey structures ranging from 14 to 17 storeys high. In two of these buildings the framing system has been used throughout the entire height of the structure but the other two have had conventional reinforced concrete frames to support the lower podium floors and the frames described here for the tower structures above. In all cases the floors used in conjunction with the frames have been conventional two way flat slabs supported by internal columns designed to resist gravity loads only but detailed to be ductile under large earthquake induced interstorey drift.

Limitations on Use

Where close spaced external concrete columns are architecturally desirable the framing system is most appropriate. It is not suitable in buildings which require large clear openings in the external structure or where internal lateral load resisting frames are needed. However because of close spaced columns, and hence many potential beam plastic hinge locations at each level, the frame system only need encompass a relatively short length of total wall length. A plan of one of the buildings in which the system has been used is shown in figure 2. The height to width ratio of each of the four lateral resisting elements is 3.51. Two thirds of the columns on the long sides are not part of the lateral load resisting system and hence no beams are required at these locations.

Architectural and Planning Advantages

In Wellington, New Zealand, where the framing system has been developed, town planning requirements dictate that to optimise the use of any given building site maximisation of the number of stories must be achieved within given height restrictions. Using conventional reinforced concrete or structural steel framing systems with low floor to floor heights means that the window head at the building perimeter is unacceptaby low. Using the framing system described interfloor heights of 2.9m have been used while still having a clear window height of 2.5m. The added advantage of a

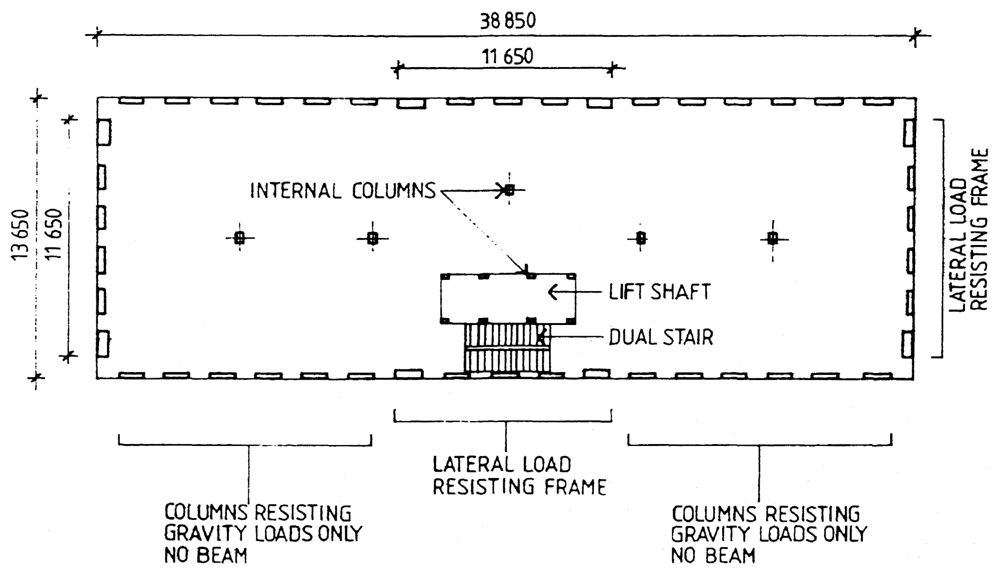

Figure 2
Plan of typical floor
showing structural system

peripheral frame means that a ceiling height of 2.55m is obtainable with no beams projecting below the ceiling. The overall advantage is an extra floor in a 15 storey building governed by an overall height limitation of 45m.

Method of Analysis and Design Actions

After performing very rapid hand analysis to determine approximate member sizes, especially for beams as architectural and construction limitations tend to govern column sizes, a two dimensional modal analysis of the building frames is carried out. The results of these analysis are checked for control of interstorey drift and any anomalies related to relative changes in stiffness which may occur as a result of storey height variations, changing floor masses etc. After appropriate adjustments to member properties static analysis of the frames are performed for horizontal and, where gravity loads are significant, vertical load cases. Because the beams in the framing system are relatively flexible it has been found that interstorey drift can be critical and thus needs to be carefully controlled. Both shear and flexural deflection and end block rigidity are taken into account in both the modal and static analysis.

From the static analysis the steel beam sections are chosen. Optimisation of the section size for each floor level can be achieved because the beams are stripped from plate and thus are not limited to stock sizes. In practice however the section depth has been varied in steps of 25mm and the width in steps of 5mm. In any of the frames designed to date a beam width of 50mm has been found to be the maximum required and 40mm has been used as the minimum width.

As the beam section properties are constant at each floor shear redistribution between the columns is necessary for compatibility. However the redistribution is not excessive and is taken care of in the column design as a result of the capacity design techniques used. The moment contribution from both the structural steel beam and the adjacent in-situ concrete slab section are factored by a 25% overcapacity factor for the calculation of the flexural action on the columns. The columns are then designed for this factored moment in conjunction with the most critical axial load, the flexural input being apportioned above and below the joint according to the New Zealand concrete design code requirements (Ref. 1).

DESIGN DETAILS

Bearing

To transfer the beam shears to the column, bearing plates are used as shown in figure 3. The concrete bearing stresses produced are high and thus special supplementary hoops are used adjacent to the plates as can be seen in the figure.

The bearing stresses were limited to those allowed in ACI 318 (Ref. 2). To ensure that the bearing plates have sufficient strength to develop the full flexural strength of the beam they are designed for uniform bearing with an overcapacity against flexural failure of 25%. Further as the concrete bearing stress is so high the centre of the bearing force on the plate is assumed to result from a uniform bearing stress, not one that is a maximum at the column face. See figure 4 for the assumed forces on the beams at the joint.

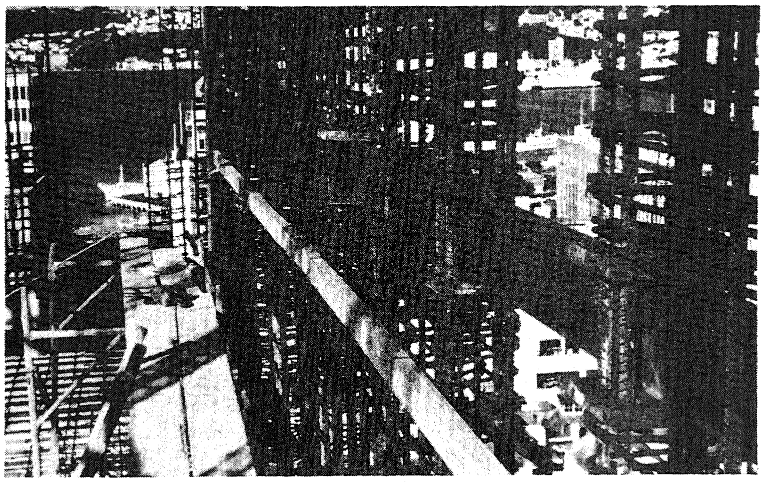

Figure 3
Typical Beam - Column Joint before the Column is Cast

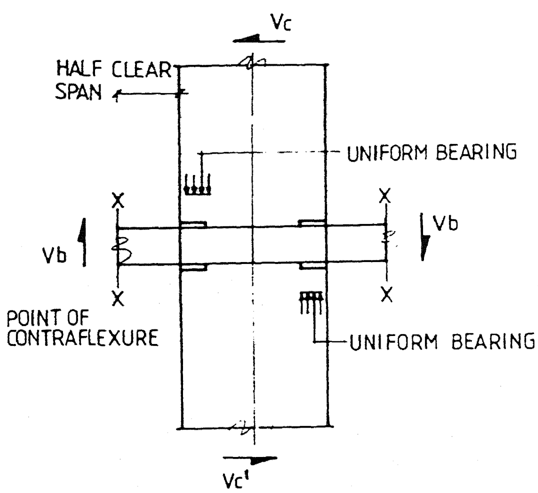

Figure 4
Forces in Column Assembly

Beam Stability

Beam stability has been checked using the criteria for rectangular sections discussed by McGuire (Ref. 3). Elastic critical moments exceed the yield moment capacities of the sections used by a wide margin. Reductions of plastic moment capacities as a result of plastic instability are of the order of one percent which is considered insignificant.

Column Reinforcement Details

Figure 3 also shows general reinforcement details in the columns. A problem which occurs because of the necessity of the beam to pass through the column is stability of bars in compression where they pass the beam. However the beams are relatively shallow and the bars must be in significant tension at the bearing plate which is on the compression face of the beam. No bars at the beam level are likely to be carrying any significant compression except in the end columns in a frame where the axial compression in the column can be high owing to overturning effects. Figure 5 shows the likely bond stress and stress distribution in a reinforcing bar in a column. This has not been confirmed with tests but the authors are confident that the bar stress distributions are realistic. To ensure stability of bars in compression in the end columns of a frame ties are welded to the steel beam to restrain the column bars.

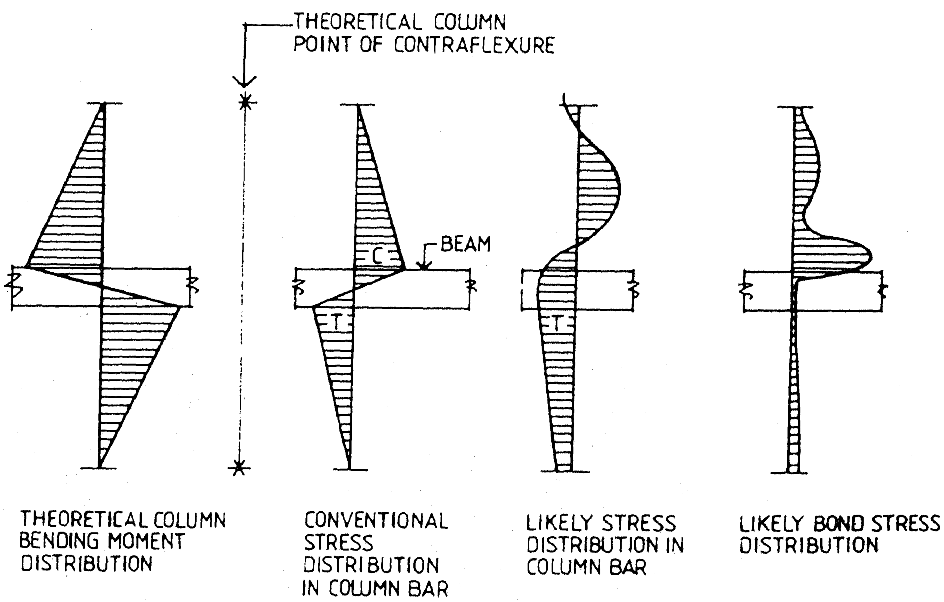

Figure 5
Comparison of Column Moment and Bar Stress Distribution

CONSTRUCTION

When first used several construction problems were encountered including difficulty in placing concrete, difficulty in consolidating concrete under the steel bearing plates, congested reinforcing in relatively narrow columns, beam placement proved to be slow especially locating long beams near other structures, crane control when placing beams, and tolerance problems with column cages. Most of the problems were attributable to lack of experience with this type of construction by the contractors concerned but some of the problem areas are worthy of further discussion.

Concrete Placing

In order to obtain sound concrete adjacent to the steel beams and bearing plates it is desirable to form the construction joint in the column some distance away from the beam. For practical purposes the construction joints have typically been 400mm above the floor slab, or 450mm above the top face of the beam. This produces a substantial departure from normal practice of casting columns to beam or slab soffit, then casting the beam and/or slab, thus forming two column construction joints at each floor. By casting the column to 400mm above the floor and then stripping the column and casting the floor slab up to the already hardened column only one construction joint is necessary in each column per floor.

However this means that the most conjested area of column is towards the top of each pour with some 2m of column to be concreted below. The conjested area contains, apart from normal column reinforcing, the beams and bearing plates plus floor slab starters and a shear key former at the column to slab interface.

Early column pours were attempted with 75mm slump concrete and 20mm maximum aggregate. Vibrating this concrete below the congested area of column proved almost impossible so the concrete mix was changed to 125mm slump and 12mm maximum aggregate. With this concrete mix a 25mm dia immersion vibrator was used and concreting the columns became much less difficult.

However the small aggregate and high slump concrete provides another problem because of its high cement content. Because the floor slab is cast after the column the contractor strips the inside shutter off the column at least a day before the outside shutter. Onset of shrinkage occurs earlier at the inside face and hence the columns tend to bow inwards up to 20mm after a few days but then gradually straighten. The Contractor thus has to adjust for column verticality at every floor to avoid the floor plan getting smaller as the building gets higher.

Placing the Steel Beams

Steel beams up to 11m in one length have been used. Where beams have been longer it has been found necessary to incorporate a site splice as longer beam lengths have been found difficult to handle in the confined sites where some of these buildings have been located. The site splice favoured is a high strength friction grip bolted type at the point of contraflexure in the steel beam. After some initial difficulties steel beams encompassing six column cages have been placed in less than an hour. The length of time to place the beams is important because it means that craneage is normally unavailable to the rest of the site during that time. Final location of the structural steel member can be done with adjustable props and crowbars. The bearing plates are already incorporated in the column cages when the beams are placed and thus help to locate and guide the placement of the beams. The bearing plates are clamped to the beams with one vertical 16mm dia bolt either side of the beam through the top and bottom plate. With experience the contractors have found the location and clamping of the bearing plates to be no problem but a careful watch has to be kept to ensure the plates are not tilted relative to the beam so that bearing occurs across the entire width of the beam edge.

COST

One of the main cost advantages in the beam and column system described is the speed of construction achieved. To some extent this is offset by the large number of concrete columns required but savings on spandrels and windows counter-balances this. Although specific cost saving is difficult to identify the system has been used in multistorey office blocks which have been built quickly and economically.

CONCLUSIONS

The system of structural steel beams and reinforced concrete columns described in this paper is an economical framing system suitable for buildings with perimeter frames, and is most appropriate in combination with flat slab construction where internal beams are undesirable. To be successfully built the system requires the building contractor to be sensitive to the requirements of well vibrated concrete in areas with congested reinforcement. Structural engineers with experience in seismic resistant reinforced concrete and steel structures would possess adequate skills for the successful structural design and detailing of the framing system.

ACKNOWLEDGEMENTS

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