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SEISMIC DESIGN CONCEPTS AND STANDARD DETAILS FOR DRIVE-IN PALLET RACK STRUCTURES

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B J Brown, E W Vautier
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A project sponsored by the Earthquake and War Damage Commission

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EXECUTIVE SUMMARY

E1: The design of Drive-In Pallet Rack Structures for seismic loads requires consideration of several secondary factors which affect both the response of the structure and sizing of members.

E2: Tensioned wire bracing systems can be an effective means of providing lateral support to the overall rack structure;

(i) Along side walls in the longitudinal direction;
(ii) Acting in conjunction with upright frames, in the transverse direction;
(iii) As a roof level diaphragm, taking longitudinal loads from near the central aisle back to the rear wall bracing.

A flexible rear plane or side wall bracing system is recommended, as allowance for increased structural deflection reduces the overall structural response of the rack to seismic loading.

E3: The design of upright frame (columns) to face loading is strongly influenced by;

(i) The range and variability of column top deflections at various positions in the rack i.e. dynamic amplification factor;

(ii) Column top and "sag" deflection under face load i.e. P-delta effects;

(iii) Amount of "fixity" achievable at column top support and baseplate level;

(iv) Whether localised "hydraulic" loading from product spillage is seen as a design condition.

E4: Factors affecting bracing design and performance:-

(i) The response of column elements to seismic loading is strongly dependent on the range and variability of the column top deflections, which comprise a roof level diaphragm and a rear plane bracing deflection component. Increased deviations of the deflected shape from the average deflection correspond to increased dynamic amplification of loadings in elements at those locations.

(ii) Rear plane or side-wall bracing provides support to the structure under longitudinal earthquake loading. A flexible rear plane or sidewall bracing system is recommended, as allowing for increased structure deflection reduces the overall response of the rack structure to seismic loading.

(iii) Tensioned wire bracing is an effective rear plane bracing system. Bracing stiffness increases with wire pretension, so the level of wire pretension in the rear plane bracing should normally be minimal. Note, however, that wires should be evenly tensioned to ensure even distribution of loading. A minimum factor of safety of 2 against wire breakage should apply.

(iv) In situations where the roof level diaphragm is supported in the longitudinal direction along one edge only, the upright frames acting in conjunction with supplementary bracing can be detailed to provide the required torsional resistance.

(v) It is recommended that, where possible, the roof level diaphragm be continuous across the centre aisle and span between rear plane or side-wall bracing lines, to minimise eccentric seismic loading effects.

E5: Factors affecting upright frame design and performance:-

(i) Design of upright frame column members in strong axis bending under longitudinal seismic face loading must take account of P-delta and dynamic amplification effects, as well as any earthquake induced axial loads. Load carrying capacity is increased by detailing for end restraints to provide at least partial end fixity.

(ii) P-delta actions have a significant effect in the design of drive-in rack structures and are not adequately provided for by reliance on the provisions of code interaction design formulae.

(iii) Columns may need to be checked for possible "hydraulic" loading, which may occur due to product spillage under earthquake loadings. Depending on the nature of product and method of storage, this effect can be severe.

E6: The magnitude of secondary effects have been inferred from trial analyses as:-

(i) P-delta actions may typically amplify column moments by about 50 to 100%, and column top reactions by about 30 to 60%. The higher amplifications would normally apply to columns with high earthquake induced axial loadings adding to gravity loads.

(ii) Typically, dynamic amplification effects only need to be considered in design of columns and those members directly supporting columns with greater than the average deflection. Loadings should be determined from an appropriate dynamic analysis or, alternatively, a dynamic amplification factor of 1.5 should be applied to the results of an equivalent static analysis.

(iii) For a typical drive-in rack structure column design moments as found from an equivalent static linear analysis may be amplified by a factor of 1.5 for dynamic effects and 1.5 - 2.0 for non-linear effects, giving a total amplification of say 2.0 to 3.0.

These indicate that a significant increase is required in the sizes of members typically used by the industry in upright frames, to comply with current seismic codes e.g. NZS4203:1984.

E7: In the design of rack systems, adequate separation should be maintained between the rack structure, and the fabric of the enclosing building, to permit predicted earthquake deformations to occur.

E8: Experimental work, and field experience has shown that:-

(i) To ensure satisfactory frame performance under lateral seismic loading, careful attention should be given to design and detailing. Experimental verification of the design is required, unless the design and detailing complies with the requirements of an approved "means of compliance" document. Tests on "standard" frames not specifically detailed for seismic loading have indicated relatively poor seismic performance.

(ii) Bracing details must be adequate to ensure proper bracing of column members in flexural and flexural-torsional buckling considerations. Bolted connections must be fully tightened for end restraints and bracing connections to be fully effective.

(iii) Column base anchorage should be designed to yield, and overstrength provided against brittle failure mechanisms. In some cases the base anchorage may be detailed to allow frames to rock under seismic loading.

E9: Notwithstanding the mandatory code provisions applying to pallet rack structures (including the effects derived from E5), the insurance industry needs to justify economically the necessity for increased standards of seismic resistance above those presently adopted by the industry. Relevant factors here include:-

(i) Risk to life (as opposed to property);

(ii) Relative cost of palletised goods, and storage rack;

(iii) Difficulty in preventing "spillage" of palletised goods in earthquakes, regardless of seismic design standards;

(iv) Temporary nature of the rack structure (as a component in a materials handling system);

(v) Damage from other sources e.g. forklifts;

(vi) Economics.

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1 INTRODUCTION

1.1 GENERAL:

There is an increasing use in New Zealand of industrial steel storage racks, or pallet racks, in line with overseas developments in materials handling technology. These racks were developed to store palletised, containerised, or large unit loads, usually placed in the rack by mechanical handling equipment.

Pallet racks may be classified into two basic types, stationary racks and portable racks, with further division into special sub-classifications by design or storage application. Racks are normally fabricated in cold formed steel, and supplied as proprietary systems. In New Zealand installations, rack heights are typically in the 6-9 metre range, with two or three elevated levels of pallets. An example of a drive-in pallet racking system with seismic bracing is shown in Fig. 1.1.

1.2 SEISMIC DESIGN ASPECTS:

Seismic design concepts for rack systems have been under investigation for some time, particularly for drive-in installations. Brown [1] has reported on United States research, and commented on the likely performance of New Zealand systems incorporating stiff bracing. The recent Edgecumbe earthquake demonstrated the vulnerability of these systems to earthquake loadings [2].

Under existing New Zealand law, pallet racks come into the category of a "Building", and require structural assessment for permit purposes including seismic loading. In an attempt to rationalise the design approach for seismic loading HERA produced in 1983 a design guide for seismic design provisions for pallet racks [3]. However, experience has shown that the nature of the industry and checking procedures are such that, in many applications, design methodology for seismic loading is not rigorous, or appropriate. There are several reasons for this:-

(i) The competitive pricing situation for racking has promoted dollar driven (as opposed to performance driven) design, i.e. any concept that adds cost is resisted.

(ii) Simple NZS4203 [4] Equivalent Static Force concepts are usually used, when dynamic analysis techniques are probably required.

(iii) Rack system operators, and approving (Local) authorities are seemingly not interested, or ignorant of the seismic design issues for what they see as <u>temporary</u> structures.

(iv) There is often little or no provision made in warehouse building design for anchoring storage rack bracing, or taking seismic load to ground.

(v) To maximise usable space, the separation between rack and surrounding structure is often minimal, and the seismic separation requirements of NZS4203 [4] are rarely considered.

1.3 STANDARD DESIGN DETAILS:

Seismic anchorage requirements vary depending on size of installation and details of support structure. However, standard components are used and manufacturers have developed their own "standard" details over the years.

Building designers need to appreciate the requirements for anchoring a rack installation, and make appropriate provision in detailing a building wherever a rack system will, or may, be installed.

Some recommendations for detailing buildings to provide for rack anchorage, together with a selection of typical standard details, are presented in a later section in this report.

The bracing details presented for discussion and analysis (refer Sections 4, 6 and Appendices) are representative of systems which have been used in New Zealand, and the conclusions of this report apply in general to all systems of this type.

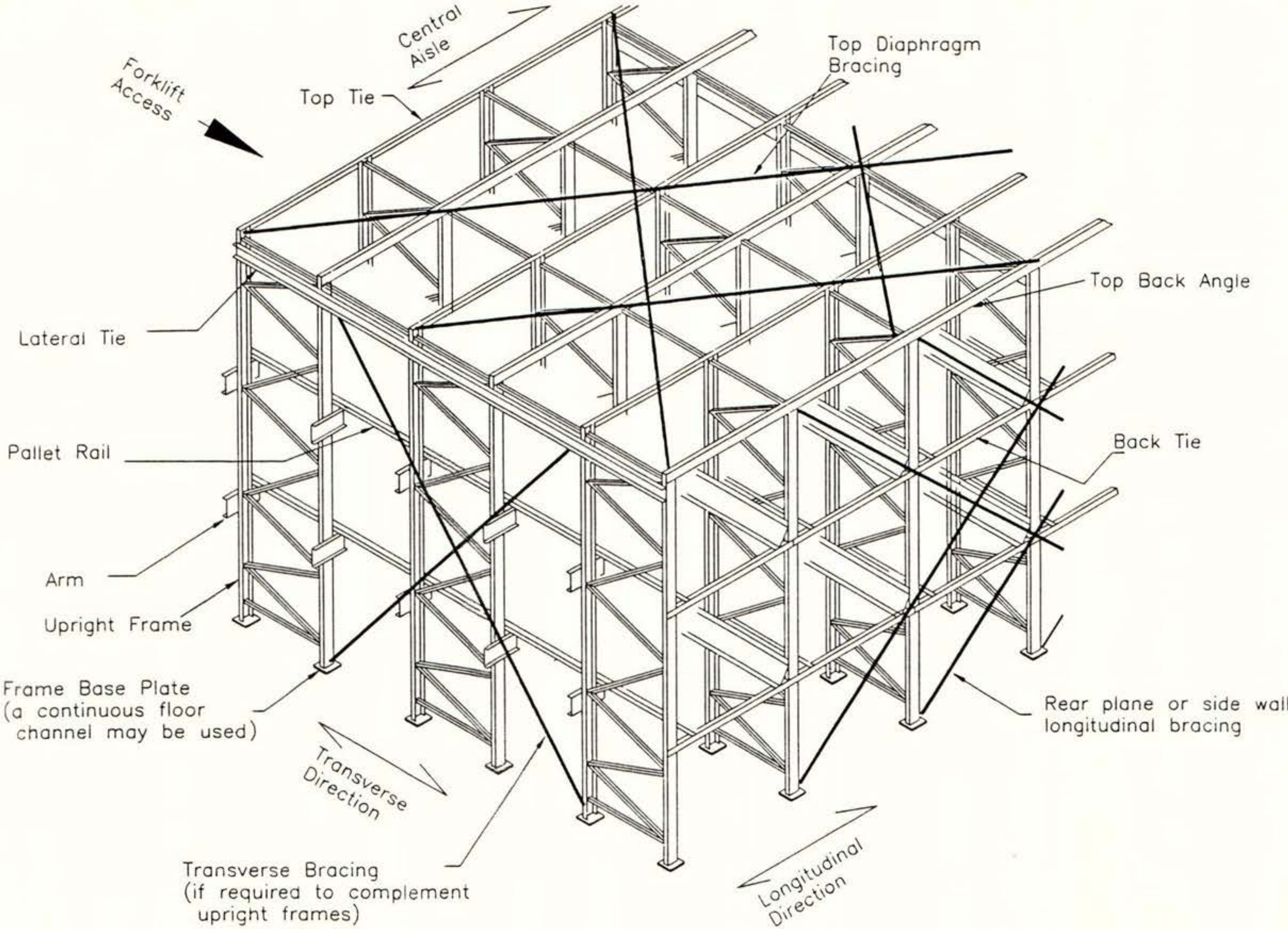


Fig 1.1 Typical Drive-in Pallet Racking System With Additional Seismic Bracing

2 SCOPE OF STUDY

This study relates specifically to drive-in rack systems, although many of the issues and concepts discussed have general application to pallet racking systems as a whole.

The work undertaken includes:-

(i) Preparation of guidelines for designers' (of both pallet racks, and the host buildings), listing essential steps and factors that must be considered in the analysis of e.g. drive-in rack with centre aisle, and external wall long-bracing. This includes model calculations incorporating results of computer simulation studies.

(ii) Preparation of standard details.

(iii) Computer simulation (linear static, dynamic and non-linear) of typical drive-in system, to determine deflections, P-delta effects and dynamic amplification factors, for different types of seismic restraint, i.e. stiff or flexible bracing.

3 DESIGN CONCEPTS AND APPLICABLE CODES

In the absence of any recognised New Zealand Standard for Pallet Racks, HERA Design Guide DG 8.3: 1983 [3] was produced by HERA with the objective of providing a unified approach to the design of pallet rack structures. This is recommended by HERA as a means of compliance with Clause 1.1.1.2 of NZS4203: 1976 (now 1984 [4]), and therefore proposed as suitable for adoption by specifiers, and authorities administering building permits. In applying the design guide it is recommended that the latest edition of the various codes and specifications referenced in it be used.

It should also be noted that DG 8.3 is directed at pallet racks in low life risk areas, which account for the majority of installations. A higher risk factor should apply in situations involving a high risk to life or property.

DG 8.3 requires adequate strength margins in design to compensate for P-Delta effects (Clause 3.4.2), and for amplification effects dependent on structure stiffness and damping and whether resonance is likely between local and global systems (Clause C3.4.4). A computer study, carried out to evaluate these effects for a typical drive-in installation subjected to earthquake loading in the longitudinal direction, is reported on in Section 6.0.

In determining the appropriate design seismic load on a rack structure, NZS3404 [5] should be used in conjunction with DG 8.3. "SM" factors for typical pallet rack structural forms are presented in Table 5P of DG 8.3. This table has been reproduced (with some modifications) in Appendix 1 to this report. Further recommended structural type factors contained in Appendix C1B of NZS3404: Part 1 complement this information. NZS3404 also gives information on design and detailing requirements to achieve the required structure ductility for a given structural type.

The "SM" factors in DG 8.3 are generally consistent with the provisions of NZS3404 with perhaps the exception of the Category C1 structures (diagonally braced upright frame with braced members capable of plastic deformation). In updating and reproducing Table 5P in Appendix 1 to this report, Category C1 has been amended to accommodate the provisions of NZS3404 for eccentrically and concentrically braced frames.

4 TYPICAL DRIVE-IN RACK DESIGN DETAILS AND STRUCTURAL SYSTEMS FOR SEISMIC LOADING

Critical aspects of drive-in rack detailing and construction, which can have a significant effect on rack performance under seismic loading, include:-

(i) Holding down bolt details;
(ii) Upright frame column details;
(iii) Lateral support system - semi-rigid frame, or bracing or a "dual" system.
(vi) Upright frame bracing systems.

A discussion of aspects of design and detailing of these elements, and influence on structural behaviour is presented in Sections 4.1 - 4.4 following.

4.1 HOLDING DOWN BOLTS:

Holding down bolts are normally proprietary mechanical anchorages, or may be grouted, and are typically anchored into a concrete ground floor slab. Anchor embedment will often be limited by slab thickness, and may not be sufficient to develop the anchor overstrength capacity. Care in design, detailing and installation is required to avoid a brittle type failure mode such as concrete cone failure or anchor pullout. References [6] and [7] provide background information and formulae for predicting the capacities of mechanical and grouted anchorages.

It is recommended that holding down bolts be designed and detailed to fail in a controlled manner, as follows:-

(i) Under lateral loading, bolts be designed to yield in shear before pullout or concrete failure occurs.

(ii) Under tensile load, bolts yield in tension and/or the tensile load in bolts be limited by an <u>identified</u> base plate yield mechanism.

The base plate yield mechanism is probably the more cost effective for these types of structures.

4.2 UPRIGHT FRAME COLUMN DETAILS:

4.2.1 General Considerations:

The load able to be carried by a slender column before a flexural or flexural - torsional buckling failure occurs depends on a number of factors, including:-

- column stiffness and cross-sectional shape;
- degree of end fixity;
- location and nature of intermediate supports
- load distribution along the column;
- sway (or translational) stiffness of the structure.

Hancock and Roos [8] conducted a theoretical and experimental study of the flexural-torsional mode of buckling of the columns forming the upright frames of a rack structure. The particular frames studied were manufactured by bolting the bracing members to the column sections, and the flexibility of the brace-column connection was found to have a significant effect on the buckling loads. The authors recommend that, where bolted connections are used, the bolts be fully tightened to ensure adequate strength.

Stark and Tilburgs [9] investigated the stability of columns in unbraced rack structures. A considerable increase in carrying capacity (in the order of 50 percent) was noted when the end condition at the floor was changed from hinges to unbolted footings on flat concrete.

4.2.2 Column Stability in Longitudinal Direction (Transverse to Upright Frames)

It is apparent that significant benefit may be had by detailing for column end restraint and allowing for this effect in design. However, the designer who allows in design for the beneficial effects of end restraints needs to ensure that these will be realised in practice. For example, some assurance is required that bolted connections will be consistently and fully tightened, and that column bases will bear evenly on a flat floor. Figure 4.2.1 illustrates how column base rotational restraint can be affected by the nature of the column to slab bearing. The column "fixing" moment may be zero (for θ less than θ_e , and has a limiting maximum value depending on column dimensions, column axial load, baseplate size and stiffness, and holding down bolt capacity.

Refer to Appendix 3 for procedures for calculating column end rotational restraints.

4.2.3 Column/Brace Design for Seismic Loading:

The work by Hancock and Roos [8] referred to in Section 4.2.1 illustrates the influence of the brace to column connection on column stability for static loads. In the case of a diagonally braced upright frame subject to seismic loading in it's own plane, rigorous design and detailing requirements apply and/or the performance of the brace to column connection must be verified by test. Refer Appendix 1, Category C for further details.

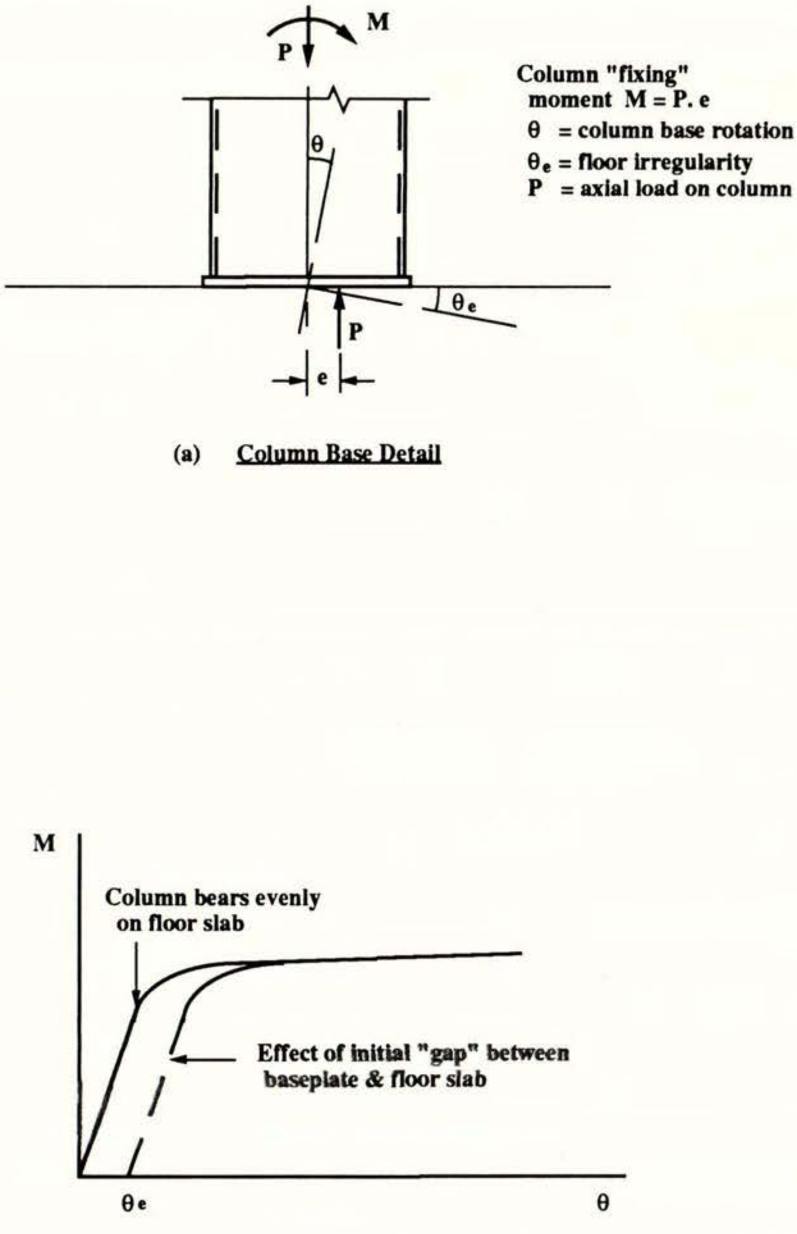


Fig 4.2.1 Idealised Column Base Bearing on Concrete Floor

4.3 LATERAL SUPPORT SYSTEMS:

4.3.1 General:

It is important to distinguish between "rigid" and "soft" lateral support systems. Rigid systems give short period structures which attract high earthquake loads. Soft systems attract lower earthquake loads but permit larger earthquake induced displacements. As a result, P-delta effects may become significant, and additional separation may be required between rack and building enclosure.

Drive-in rack systems typically comprise linked upright lattice frames in the transverse direction. In the longitudinal direction, frames span between the floor and a "roof diaphragm", which may be a series of trusses or diagonal bracing elements designed to transfer load to the rear face of the racking system, which itself is normally braced in the longitudinal direction. Refer Figs. 1.1, 4.3.1 and 4.3.2 for typical details. Some installations incorporate longitudinal anchor frames at the rear face which may be used with or without longitudinal bracing.

The bracing shown on Figs 4.3.1 and 4.3.2 is representative of systems which have been used in New Zealand, and the following discussion in Sections 4.3.2 to 4.3.4, and later analysis (Section 6 and Appendices), while referenced to the system shown, applies in general to all systems of this type.

4.3.2 Transverse Bracing Direction:

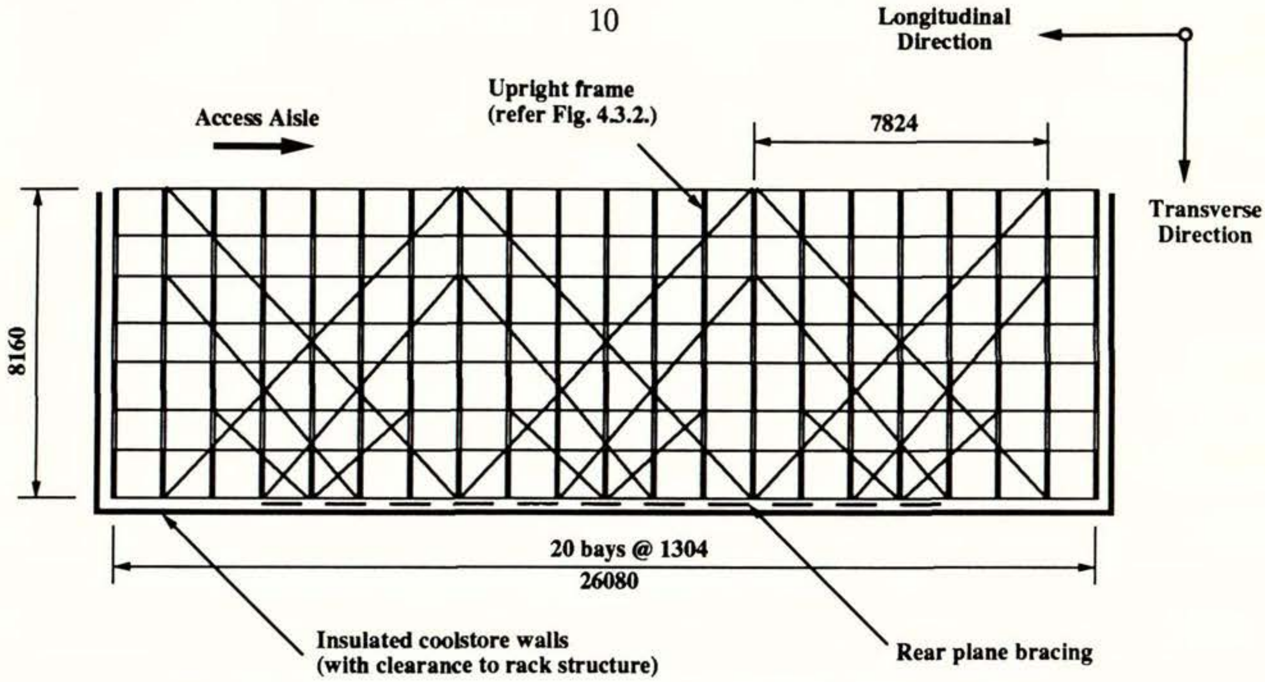
A typical transverse frame installation consists of a series of upright frames linked together, as shown in Fig. 4.3.2. Each frame is braced diagonally and a stiff system results which is often vulnerable to damage at joints. The maximum loads in frame members may be controlled by allowing frames to rock under lateral loading [1].

Tensioned wire bracing may be used in conjunction with frame action to give a "dual" system, where frame action resists at least 25% of the total seismic load. In design, frame stiffness and bracing stiffness must be determined in order to apportion seismic loading to each system.

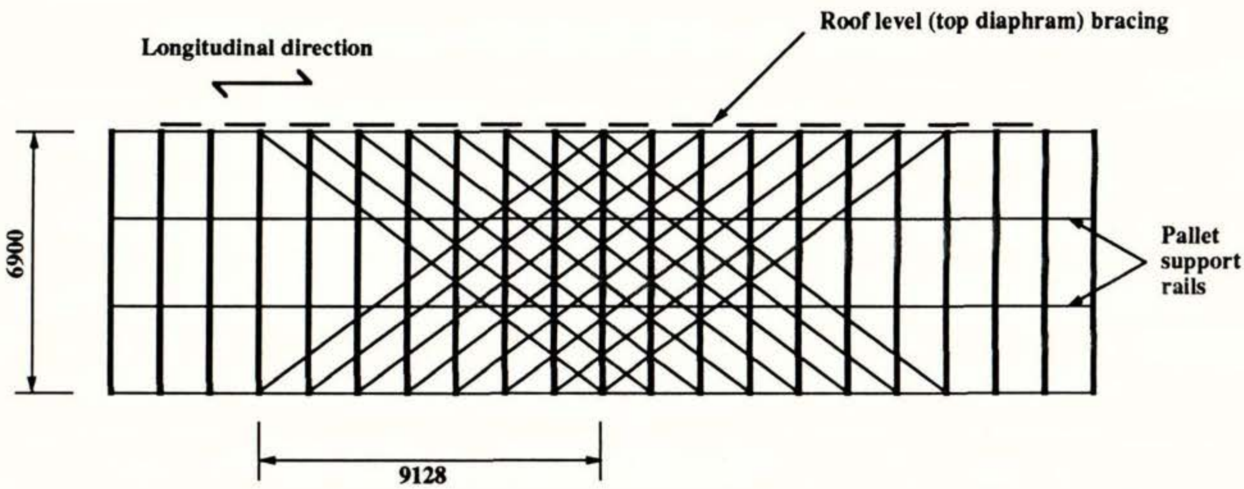
4.3.3 Longitudinal Bracing Direction:

Up until about 1983 most installations in New Zealand used some form of mild steel bracing to provide seismic restraint in the longitudinal direction. This resulted in relatively stiff structures, attracting high accelerations (and loads) and thereby promoting load spillage in only moderate earthquakes. A further disadvantage was the concentration of lateral loads into relatively strong bracing elements leading to associated difficulties in design and detailing to effectively connect the bracing to the light thin walled sections traditionally used in rack structures.

The recognised disadvantages of mild steel bracing led to the development of a form of bracing known as "wire basketing", first used in a rack structure in New Zealand about 1979. The system utilises a grillage of tensioned wires or cables,



Plan - Showing Roof Level (Top Diaphram) Bracing
(Note - Rack storage on other side of central aisle similar)



Rear Elevation - Showing Rear Plane (Side Wall) Longitudinal Bracing

Fig. 4.3.1. Drive-in Rack Layout In A Coolstore Showing A Typical Bracing Arrangement

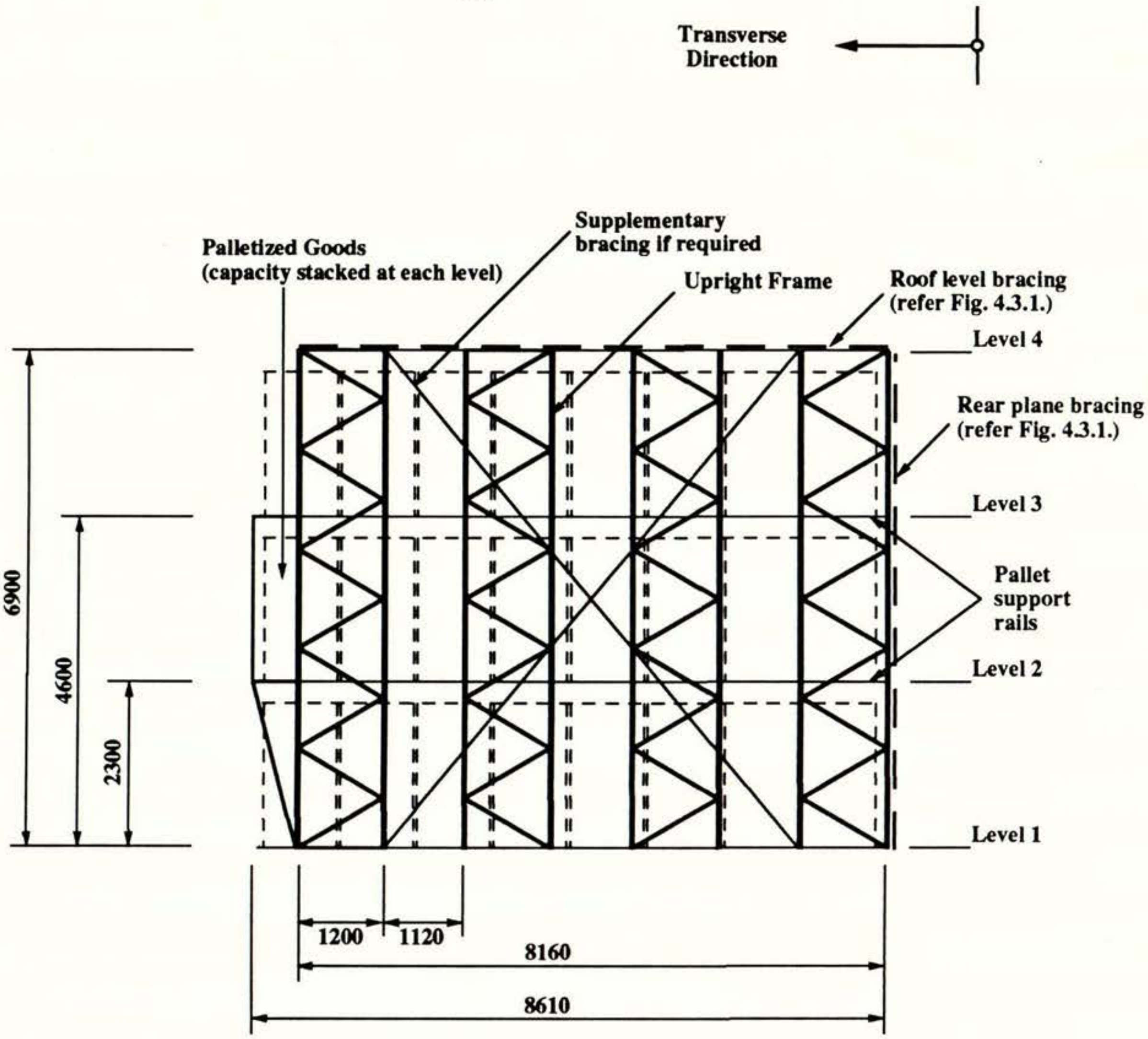


Fig. 4.3.2. Side Elevation of Drive-In Rack showing Upright Frames and Transverse Bracing

usually in both the top diaphragm and in the rear bracing panel, refer Fig. 4.3.1 for typical layout details. The use of wire rope or high strength steel tendons allows increased displacements as compared with mild steel bracing, and can attract significantly less loading due to increases in structure period, as well as increased damping associated with structural deformations.

The advantages of a "wire basketing" system are that it has intermediate stiffness and provides for good distribution of applied lateral loadings. Disadvantages are an ongoing maintenance requirement and (possibly) vulnerability of the small wire section to damage. Refer to Section 4.3.4 for further details and notes on design of wire basketing systems.

While "wire basketing" is currently the form of bracing most used for rack structures, this does not preclude the use of other systems, particularly where yielding elements and/or energy dissipators may be incorporated. Examples of devices which have been proposed for energy dissipation include braced frame energy absorbers [10, 11] and friction damped braced frames [12].

Where a flexible bracing system is used, it is advantageous in design to take account of framing action which may arise from top tie and back tie members with rigid or semi-rigid connection to columns. If the frame action resists at least 25% of the seismic load, the system may be designed as a "dual" system. In the case of the RACK10 model analysed in this study (refer Section 6.5), frame action resists about 10% of the seismic load.

4.3.4 Wire Basketing Systems - Design Notes:

The potential for sudden failure (wire breakage) may be minimised by following good design, installation and maintenance procedures. These include:-

(i) Selection of materials and corrosion protection suitable for the particular application;
(ii) Use of an appropriate factor of safety in design;
(iii) Attention to detail, especially of end connections;
(iv) Implementing a regular programme of inspections.

4.3.4.1 Bracing Wire:

Bracing wire may be high strength tendon, stranded cable or wire rope. Commonly used bracing materials in New Zealand applications include 7 mm BBR prestressing wire and 10 mm wire rope. For a given design strength, these materials are considerably more "elastic" than mild steel rod (i.e. lower EA value) and provide for greater deflections of the rack structure. Refer Appendix 2 for an example calculation comparing different bracing systems.

4.3.4.2 **Factor of Safety:**

The permissible wire/cable design load is normally determined from it's ultimate tensile strength by applying an appropriate factor of safety (FOS) which should be assessed taking into account the load case and environment. According to [13] the minimum FOS under dead load + prestress + earthquake should be 2.0, assuming an interior benign environment. A higher FOS should be considered for exterior and/or corrosive environments. Temperature effects should also be taken into account.

4.3.4.3 **Top Diaphragm Bracing:** (refer Fig. 4.3.1(a))

In design of top diaphragm bracing elements, individual element loadings will depend on relative element "stiffness" as well as load distribution in plan, both of which may be difficult to predict in practice. The lateral seismic load is likely to be weighted towards the centre aisle, due to dynamic amplification effects. Also, the pallet loading is unlikely to be evenly distributed. The effective bracing "stiffness" will depend on brace length and level of prestress, as well as stiffness of chord elements and upright frames.

It is recommended that design of plan bracing take account of actual load distribution due to structural form, as well as allow for a loading eccentricity of ± 0.1 B as a minimum.

4.3.4.4 **Rear Plane or Side Wall Longitudinal Bracing:**

For commonly used arrangements (such as that shown in Fig. 4.3.1(b)) lateral loading will be evenly shared between brace elements, provided that braces are uniformly prestressed. This should be a specified requirement of the installation.

4.4 UPRIGHT FRAME BRACING SYSTEMS:

4.4.1 General:

Conventionally braced frames may not perform well under seismic loading. Chen and Scholl [14] report that a suitably high load factor or some design modifications to braced frame systems are needed to preclude early nonductile damage during a strong earthquake. They conducted an analytical study to investigate the feasibility of an eccentric bracing system as a means of achieving the desired seismic performance, with promising results. The analytical study found that an eccentrically braced frame with pinned beam-column connections could undergo sizable amounts of inelastic deformation without suffering structural instability when subjected to an earthquake motion corresponding to a peak ground acceleration (PGA) of 0.3 g. By comparison, the conventional bracing system could be expected to undergo considerable nonductile damage, implying brace buckling failure at a very low level of earthquake excitation corresponding to a PGA of less than 0.1 g.

4.4.2 Eccentrically Braced Frames:

A design method for Category 1 (fully ductile) eccentrically braced frames is detailed in Clause 12.11 of NZS3404: Part 2 [5]. Compliance with this design procedure would require the use of members not traditionally used in racking systems. For example, beams must comply with the requirements for Category 1 members. For this reason, experimental verification is recommended (refer Section 4.4.6).

4.4.3 Concentrically Braced Frames:

A design procedure for concentrically braced frames is detailed in Clause 12.12 of NZS3404: Part 2 [5]. Compliance with this design procedure would require a significantly larger brace section than is current practice in upright frame construction (refer Design Example, Appendix 5). It is worth noting that this observation is in line with the results reported on by Chen and Scholl [14].

4.4.4 Base isolation Through Rocking Response:

As reported by Brown [1] seismic load limitation may be achieved by allowing frames to rock. Special design considerations apply, and column bases need to be suitably detailed to permit rocking to occur.

4.4.5 Mixed Seismic-Resisting Systems:

Wire bracing may be used in parallel with upright frames in resisting seismic loads. The design load should be shared between the two systems on the basis of their relative stiffnesses. The appropriate S factor should be determined for each system independently and the final design forces determined in accordance with the recommendations of NZS4203 [4], Clause C3.4.2(b).

4.4.6 Experimental Studies:

Designs not conforming to accepted design and detailing rules require experimental verification in accordance with Clause 4.1 of DG 8.3 [3].

5 DESIGN OF TYPICAL DRIVE-IN RACK INSTALLATION FOR SEISMIC LOADING

5.1 GENERAL

Drive-in racks are typically designed for seismic loadings using simplifying assumptions and methods which may not be appropriate for what are relatively complex structures. A non-linear dynamic three dimensional analysis package will give the best assessment of element loadings, provided that the structure is correctly modelled. Alternatively, an equivalent linear-static analysis may be used, introducing factors to account for non-linear and dynamic effects, and employing a degree of judgement to compensate for the less detailed analysis. Whichever method is used, care must be taken to ensure that the analysis model adequately represents the actual structure.

Some design notes are included in Section 5.2 following, and a design procedure based on the equivalent linear-static analysis method is presented in Appendix 5. Refer also to Appendix 6, reproduced from Appendix D of DG 8.3 [3], for a schedule of essential information to be completed by rack purchasers/specifiers.

5.2 DESIGN ASPECTS

5.2.1 Rack Geometry:

The rack geometry will normally be fixed by owner requirements and building constraints.

5.2.2 Structural Systems:

Select the structural systems for resisting lateral seismic loading, in both the transverse and longitudinal directions. A preliminary assessment of bracing requirements and member sizes must be made, which will normally be based on rack loading and previous experience. Some preliminary first order calculations may assist in selecting member sizes for a more detailed analysis.

5.2.3 Rack Loading:

The rack design loading will be fixed by owner requirements. In some cases loadings will be well defined, but in other applications it may be necessary to make an assessment of likely maximum loads. Wherever possible the designer should inspect existing facilities handling products of the same type, to ensure he fully appreciates likely operating procedures and loading conditions.

5.2.4 Determine Column Allowable Axial Load:

This will include calculation of effective lengths, which may take into account column end rotation restraints, where applicable. Bracing stiffness should be checked to ensure that the column is in effect "sway-prevented", although this will usually be found to be the case.

5.2.5 Seismic Loads:

Seismic loading depends on locality, structure type and stiffness and soil type. Location in building and building type will also affect the level of seismic load if the rack structure is not directly supported on a ground bearing slab.

NZS4203 [4], NZS3404 [5], and DG 8.3 [3] all provide guidance on calculation of seismic loading for a given application.

5.2.6 Design for Lateral Seismic Loads:

The structural system for drive-in racks loaded in the longitudinal direction comprises vertical columns in major axis bending, supported at their tops by a roof plane diaphragm or bracing system acting in conjunction with side wall bracing and/or anchor frames.

In the transverse direction, the structural system typically comprises upright frames, with or without supplementary bracing.

The following factors will normally apply in design:-

(a) For an equivalent static analysis, distribution of horizontal seismic forces should be undertaken in accordance with Clause 3.4.6.1 of NZS4203 [4];

(b) Where both bracing and frame action occur together, loads should be shared between the two systems in proportion to their stiffnesses;

(c) The earthquake induced load in exterior columns is increased due to pallet centres of mass above rail levels (refer Figs. 5.2.1 and 6.3.1).

(d) In most applications a reduced gravity load of 75% of the maximum storage load (Clause C3.3 in DG 8.3 [3]) may be used in assessing the lateral seismic loading for design of the following elements:-

    (i) The roof level diaphragm;

    (ii) The rear plane bracing;

    (iii) The upright frames (since the lateral load on any line is shared between a number of frames and there would also be a certain amount of load redistribution from heavily loaded to lightly loaded lines).

(e) On a local scale, the full gravity load will apply for the assessment of the following design loadings:

    (i) Gravity loading in the upright frame column legs;

(ii) Bending moments and deflections in column legs for lateral earthquake loading in the longitudinal direction (but note that column top deflection may be limited to that corresponding to the "global" level of loading).

(f) Drive-in rack structures will often be symmetrical about a centre aisle, and in such cases the top diaphragm may be connected across the centre aisle to provide a symmetrical structure with respect to lateral earthquake loading (both longitudinal and transverse). In some situations this symmetry is not available, or no cross-aisle connection is provided (as in the structure shown in Fig. 4.3.1). In this case, the line of action of the longitudinal loading is eccentric to the load resisting system i.e. the rear plane (or sidewall) bracing. The upright frame lines (with or without supplementary bracing) must provide the moment couple required for overall structure equilibrium (as indicated in Fig. 6.4.2, for example). This case may control the design of the upright frames.

(g) Design of the roof level diaphragm should allow for a loading eccentricity of ± 0.1 B as a minimum (refer Section 4.3.4.3).

(h) Under longitudinal earthquake loading, column lines deflect by different amounts depending on location and structure details. Dynamic amplification of moments and shears occurs in the columns with the higher deflections. A dynamic analysis is required, or a dynamic amplification factor of at least 1.5 (refer Section 7.0) should be applied to the following design loadings calculated in a static analysis:-

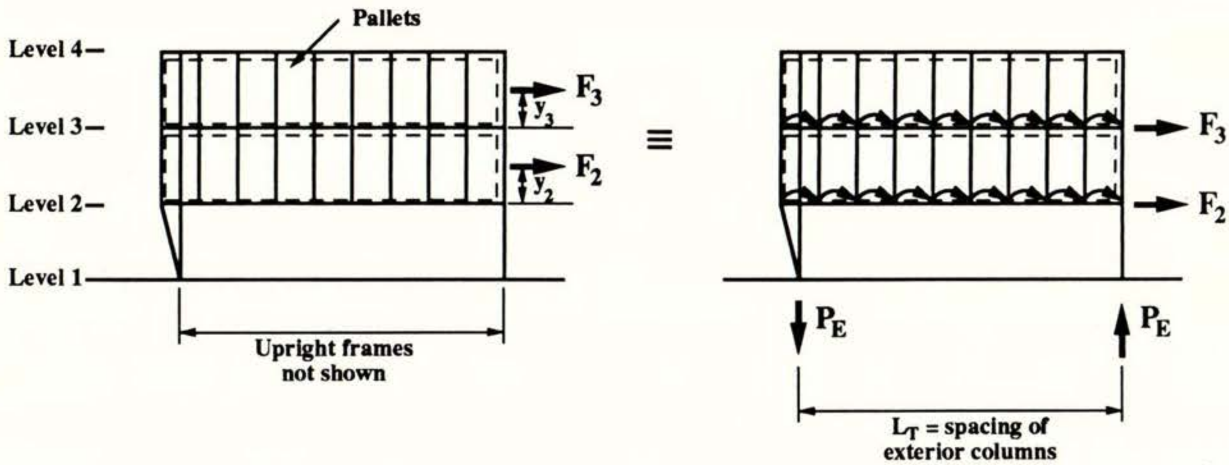
(i) Column bending moments and shears (in strong axis bending).

(ii) Forces in top diaphragm elements supporting the column lines with greater than average deflections.

(i) P-delta effects should be allowed for as discussed in Section 8.0.

(j) Where seismic loads are limited by inelastic behaviour, adequate overstrength in non-ductile elements must be provided.

(k) Column anchorage must be designed to resist column base shear and uplift forces. Detailing should provide for a ductile failure mode, and overstrength provided against brittle failure mechanisms. Special detailing provisions may be included, for example to allow frame rocking with consequent limitation of loading on upright frame members.



Notes:

(1) Pallet centres of mass are above rail level, and lateral forces produce vertical reactions along the pallet support rails, as well as horizontal forces. Refer diagram

These vertical reactions :-

a) Cancel out at interior columns

b) Produce additional load in exterior columns given by

$$P_E = (\Sigma F_x y_x) / L_T$$

(2) Frames are analysed for horizontal forces applied at rail level, and P_E is added to the resultant forces in exterior columns.

Fig 5.2.1 Lateral Loading Of Upright Frames

6 COMPUTER MODELLING AND ANALYSIS

6.1 INTRODUCTION:

Computer modelling and analysis work undertaken as part of this project had the objectives of:-

(i) Verification of assumptions made in a linear static analysis;

(ii) Investigation of dynamic amplification effects and sensitivity of the structural response to variations to the structure parameters.

A general purpose finite element analysis computer program, LARSA [15], was used for all computer modelling work carried out in this study. LARSA is both PC and VAX based and has non-linear and dynamic analysis capability.

Various models were formulated and analysed to provide the required data, as described in Sections 6.2 to 6.5 following.

6.2 BRACED UPRIGHT FRAME MODEL:

6.2.1 Model Geometry:

A typical upright frame which might be used in a rack installation such as that shown in Fig. 4.3.1 was modelled. In line with the recommendations of Chen et al [16], a reduction factor of 7 was applied to the cross-sectional area of the lacing. The model, named UFRAME, is detailed in Fig. 6.2.1.

6.2.2 Analysis Cases:

The following analyses were carried out:-

(a) Eigen Value;
(b) Equivalent Static Analysis (ESA);
(c) Response Spectrum Analysis (RSA).

For the ESA, horizontal seismic forces were distributed in accordance with Clause 3.4.6 of NZS4203 [4]. The corresponding lateral forces, as derived in the design example (Appendix 5), are shown on Fig. 6.2.1. The linear static analysis run included a second load case, consisting of a 10 kN lateral load applied at the top of the frame. This additional loading case gave a frame "stiffness" value for use in the analysis of the roof level diaphragm (Section 6.4).

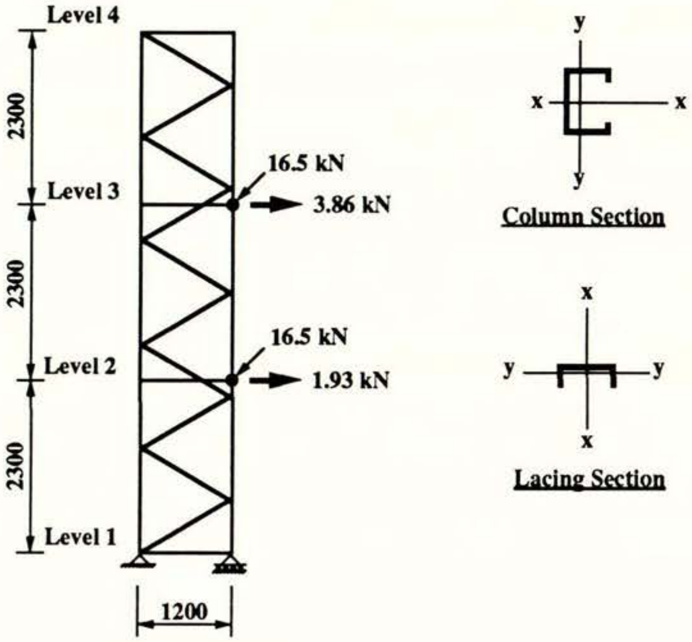
Loadings for the eigen value and RSA analyses consisted of weights of 16.5 kN lumped at pallet rail levels (representing a reduced gravity load - refer to the design example in Appendix 5).

6.2.3 Results:

A summary of output data is presented in Table 6.2.1. As expected for this model, results from the ESA and RSA runs were in good agreement. The RSA output data has been scaled to correspond to a base shear of 100% of that for the ESA, to permit a direct comparison of the results.

UFRAME ANALYSIS RESULTS SUMMARY		
ESA and RSA Analyses		
	ESA	**RSA**
Displacements (mm)		
Level 4	10.7	10.3
Level 3	9.9	9.6
Level 2	5.9	5.9
Base Shear	5.79 kN	5.79 kN
Earthquake Induced Vertical Reactions	±18.5 kN	±17.9 kN
Frame "Stiffness"		
Top horizontal loading/unit deflection = 290 kN/m.		
Eigen Value Analysis		
Mode	Natural Frequency (c/sec)	Period (sec)
1	2.37	0.42
2	6.46	0.15

Table 6.2.1 - Summary of UFRAME Analysis Output Data.



Elevation of model "UFRAME"

Member Properties

	A (mm^2)	I_x (mm^4)	I_y (mm^4)
Column	854	2.08×10^6	0.46×10^6
Lacing*	38	0.02×10^6	0.002×10^6

* Lacing properties reduced by a factor of 7 (after Chen et al [16])

Fig 6.2.1 2D Braced Upright Frame Model

6.3 COLUMN MODEL:

6.3.1 Model Geometry:

A simple two dimensional model was prepared to represent the performance of the drive-in rack installation shown in Fig. 4.3.1 under the action of longitudinal loading. The model, named COL10, is detailed in Fig. 6.3.1. It comprises a vertical column, together with rotational and translational springs to model partial base fixity, top tie beam and bracing stiffness. The procedures for calculating rotational spring stiffness values are described in Appendix 3. Translational spring stiffness varies depending on column location, and the example calculation in Clause A3.4 of Appendix 3 details a procedure for calculating minimum and maximum values for the translational spring stiffness, which bound the problem.

The moment couples m_x are included in Fig. 6.3.1 to show how pallet lateral loads may be taken to act at the pallet rail level. These moment couples may be omitted from the column analysis, as they do not significantly affect the final design. For typical configurations, design loads will be within about 10% of values obtained if moment couples are included.

6.3.2 Model Analysis:

The following analyses were carried out:-

(a) Eigen Value;
(b) Equivalent Static Analysis (ESA);
(c) Response Spectrum Analysis (RSA).

For the ESA, horizontal forces were distributed in accordance with Clause 3.4.6 of NZS4203 [4]. A series of analysis runs were completed, for a range of end restraint conditions.

6.3.3 Results:

Results of the ESA and RSA are shown on Figs. 6.3.2 and 6.3.3 for various column end restraints as shown, and summarised in Table 6.3.1. The RSA data has been scaled, such that the sum of the top and bottom column shears is equal to the design earthquake force used in the ESA. This allows direct comparison of the output from the two alternative methods of solution.

Comparison of the RSA and ESA output data indicates that redistribution of horizontal forces does occur, but not to the extent given by Clause 3.4.6 of NZS4203 [4] for buildings. This is to be expected, due to the effect of column end restraints on the deflected shape of the column, because there would be no force re-distribution if a no-sway fully pinned column end condition was assumed. While the ESA method with NZS4203 load redistribution is only approximate for this application, results from the ESA and RSA methods generally agree to within about 10% for the cases considered. This is not a large discrepancy compared with the effects of other factors involved, such as end restraints, P-delta actions and the differing dynamic response of columns throughout the rack structure (refer Sections 6.4 and 6.5).

Referring to the summary of output data in Table 6.3.1, variation of column end restraints produces a 20 to 26% variation in column horizontal end reactions, and a 37 to 50% variation in bending moment along the length of the column. Clearly, variations of this magnitude are significant in terms of design choices. The maximum (or close to maximum) values for column top reaction (affecting bracing design), and bending moments (influencing upright column sizing), are obtained from the pinned ended analysis case.

COL10 ANALYSIS RESULTS SUMMARY (a)						
ESA and RSA Analyses	ESA			RSA		
	Min.	Max.	Max. Min	Min.	Max.	Max. Min.
End Horizontal Reactions (kN):						
Level 4	1.52	1.89	1.24	1.46	1.77	1.21
Level 1	1.41	1.78	1.26	1.53	1.84	1.20
Bending Moments (kN-m)						
Level 4	-	1.25	-	-	1.23	-
Level 3	3.09	4.22	1.37	2.84	4.02	1.42
Level 2	2.24	3.37	1.50	2.53	3.67	1.45
Level 1	-	1.70	-	-	1.66	-

Eigen Value Analyses

Column End Restraint (refer Fig 6.3.1)			First Mode Response	
K_{z1} (kN-m/rad.)	K_{z4} (kN-m/rad.)	K_{x4} (kN/m)	Natural Frequency (c/sec)	Period (sec)
0	0	10	.50	2.00
50	0	10	.55	1.81
50	180	10	.55	1.81
0	0	50	.75	1.33
50	0	50	.81	1.24
50	180	50	.85	1.17

Table 6.3.1 - Summary of COL10 Analysis Output Data (a)

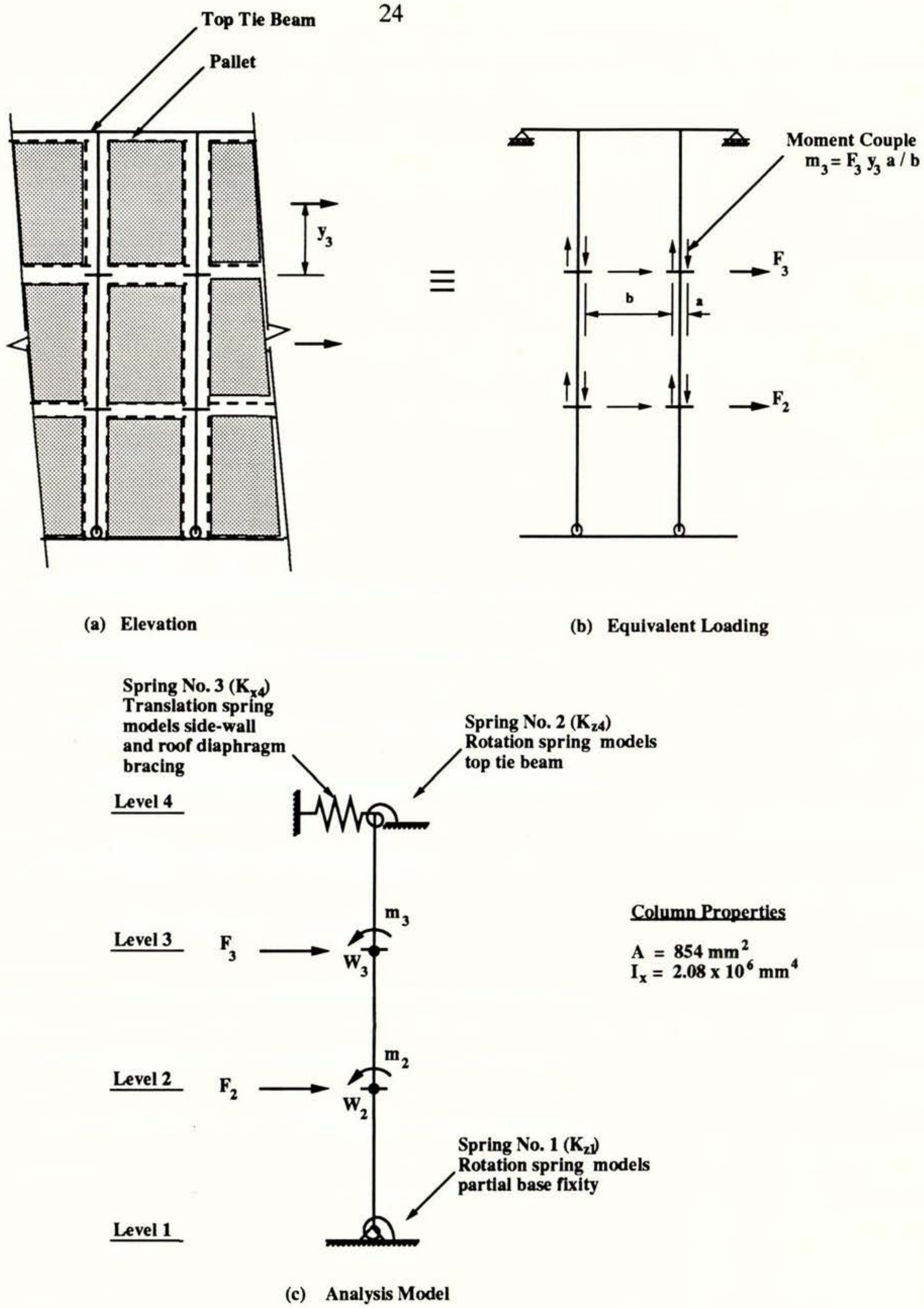


Fig. 6.3.1 Typical Interior Column Under Longitudinal Earthquake

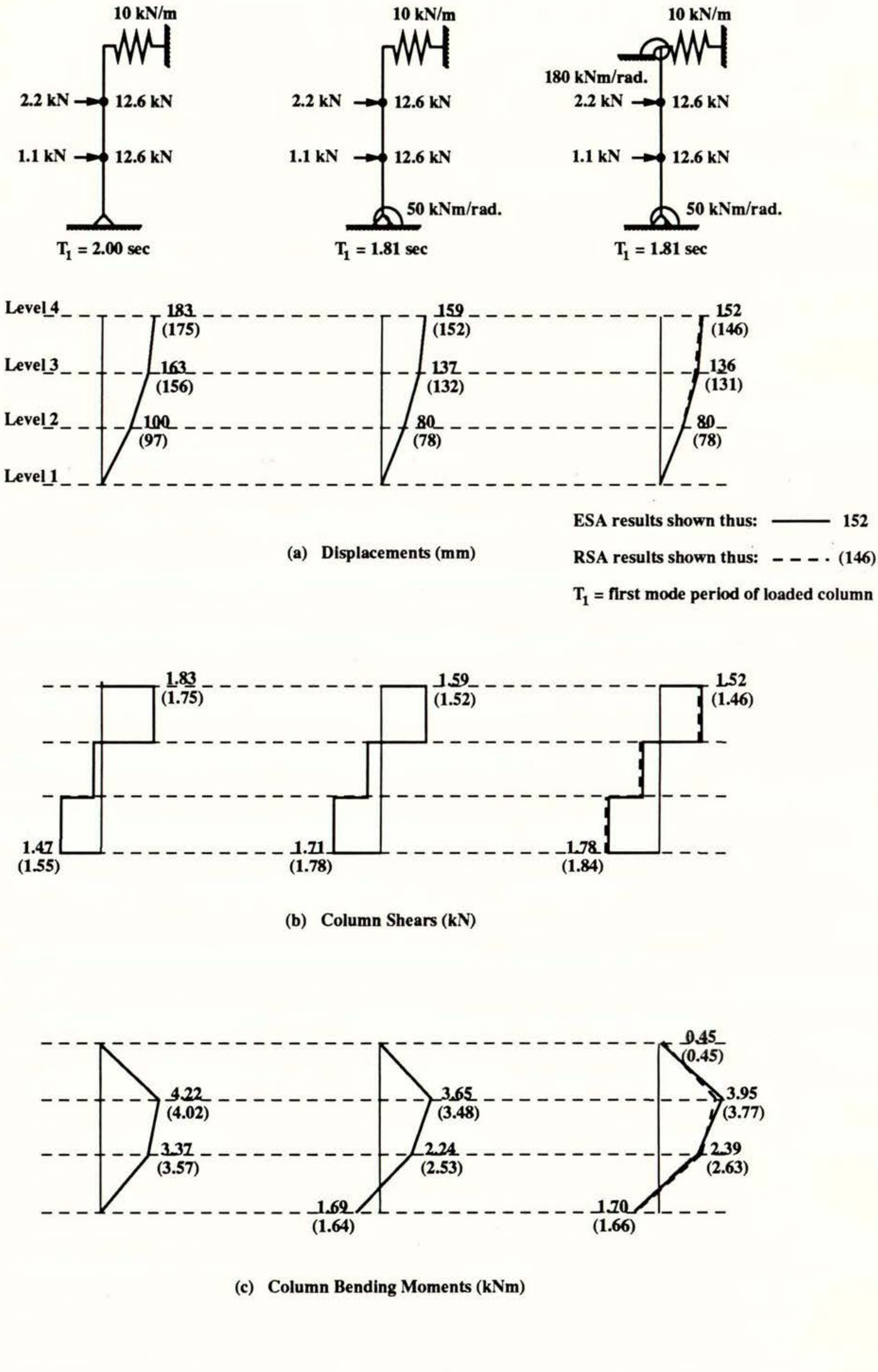
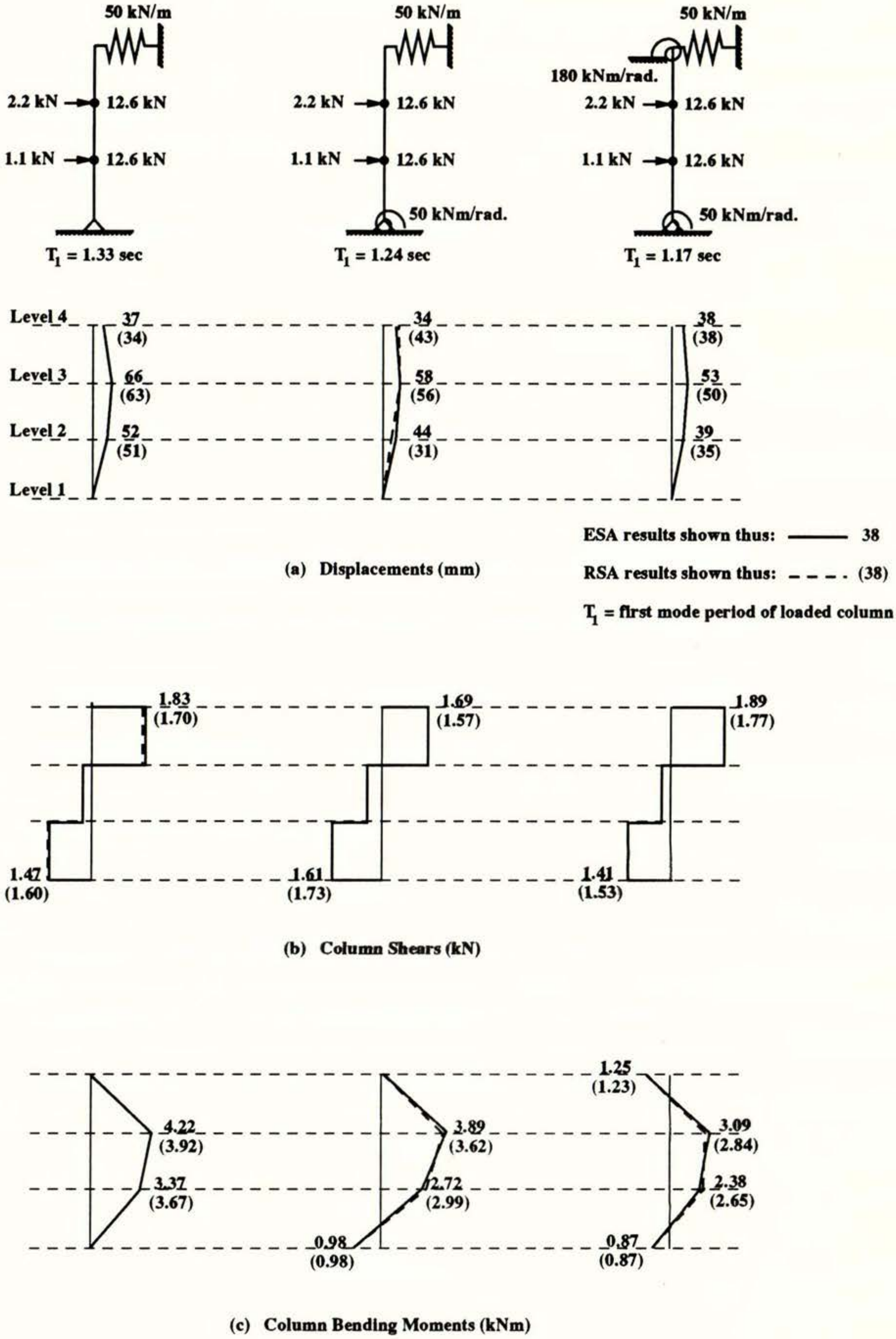


Fig. 6.3.2 COL10 Output Data For Various End Rotational Restraints and K_{x4} = 10 kN/m



(a) Displacements (mm)

(b) Column Shears (kN)

(c) Column Bending Moments (kNm)

Fig. 6.3.3 COL10 Output Data For Various End Rotational Restraints And K_{x4} = 50 kN/m

6.4 ROOF LEVEL DIAPHRAGM:

6.4.1 Model Geometry:

To reliably determine column top deflections, and the distribution of load into bracing elements, a realistic model of the roof level diaphragm is required for analysis. In this study, the roof level diaphragm was modelled as shown in Fig. 6.4.1. Member properties have been modified to take account of joint details while keeping the model as simple as possible. The modifications are as follows:-

(a) Longitudinal tie members (LTIE) have been given an artificially low lateral bending stiffness ($I_y = 0.01 \times 10^6 \text{ mm}^4$). The lateral bending stiffness of the top back angle (BTIE) has been increased to compensate.

(b) Transverse tie members (TTIE) are pinned at their junction with the top back angle on Line 1.

(c) A uniform distribution of TTIE members along the rack has been assumed for the purpose of this analysis while, in practice, these members may vary along the length of the rack. The assumption is valid provided that the total lateral bending stiffness is not altered.

Modifications (a) and (b) in effect model flexible connections between longitudinal and transverse members, while allowing that continuous members are provided in each direction. It is considered that this would represent a typical roof level diaphragm construction, and different modelling techniques will be appropriate for other forms of construction.

The upright frame restraints are modelled as grounded springs of stiffness K = 1,000 kN/m (from Section 6.2.3 the frame stiffness is 290 kN/m, giving say 1,000 kN/m for 4 frames linked together). The rear plane bracing has been modelled as a grounded spring of stiffness K = 4,800 kN/m (say 8 wires at 600 kN/m - refer Appendix 3).

6.4.2 Model Analysis:

The model was analysed for a static longitudinal loading of 190 kN (refer Appendix 5 for derivation), distributed as shown in Fig. 6.4.2. The following analyses were carried out:-

(a) A linear analysis, with all "compression" brace members removed from the model.

(b) A nonlinear analysis, using cable elements for the bracing members. These were given an initial pretension of 10 kN, and are effectively removed from the model if they go into compression.

6.4.3 Results:

Brace loads, "reactions" and frame deflected shapes obtained from the non-linear analysis are shown in Fig. 6.4.2. Results from the linear analysis were within 10% of these values, and would have been even closer if brace area had been increased to model the higher effective stiffness provided by a pair of diagonally opposed pretensioned wires (refer to Appendix 2 for an example illustrating this).

Referring to Fig. 6.4.2 the maximum brace load is 31.9 kN which is about 25% higher than the average loading. It is apparent, therefore, that it would be unconservative to assume that loads are shared evenly between bracing elements. Also, design should allow for an eccentricity of weight distribution which would give a further increased variation in brace loadings.

The "reactions" shown on Fig. 6.4.2 (a) are, in effect, lateral loadings applied to the tops of the upright frames. Loadings vary along the length of the rack, with a maximum of 24.8 kN (applied to a line of 4 upright frames). Loads would be more evenly distributed if, for example, the lateral bending stiffness of the top back angle and/or longitudinal ties was increased relative to upright frame stiffness, and vice versa.

The use of tensioned cross-bracing on some lines would also effectively "stiffen" the top diaphragm, although the "reaction" loads on braced lines would be increased. However, these loads would be shared between upright frames and cross-bracing in proportion to their stiffnesses.

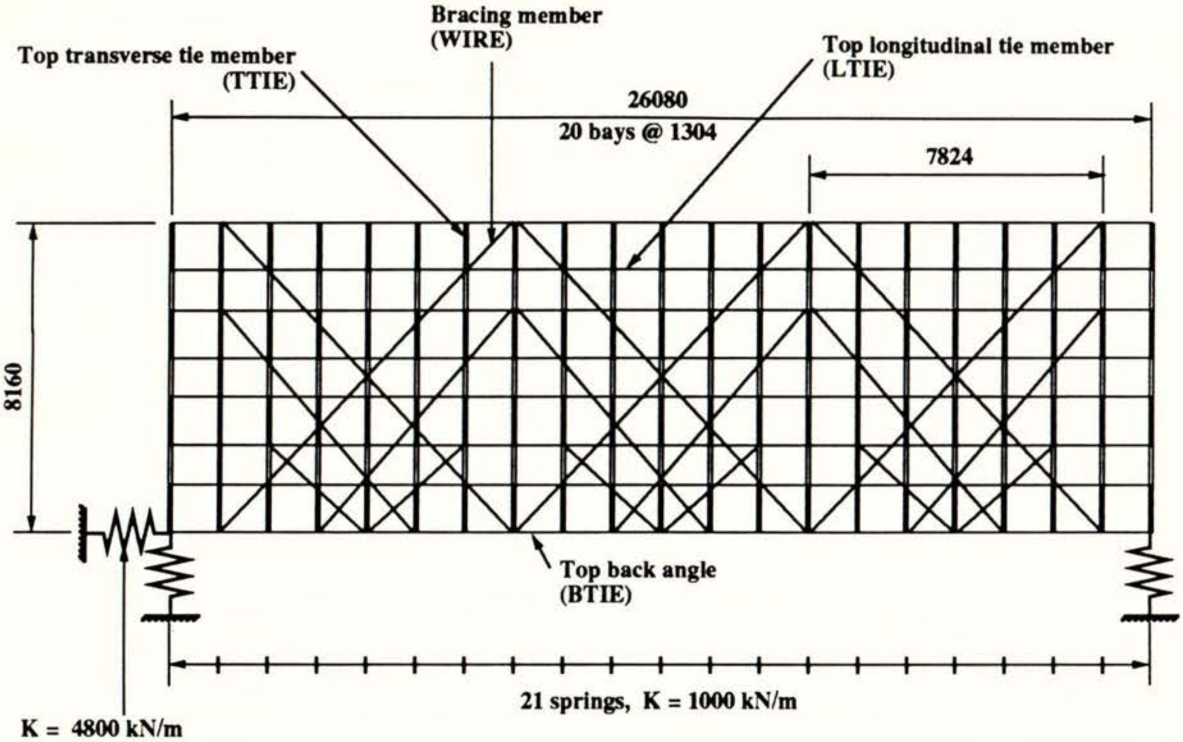
The deflected shapes shown in Fig. 6.4.2 (b) provide a key to assessment of dynamic amplification effects. Increased deviations of the deflected shape from the average deflection correspond to increased dynamic amplification of loadings in elements at those locations.

Thus, improved rack performance would result from:-

(a) Providing for an increase in the rear plane bracing proportion of total deflection, with a maximum deflection limited by stability and practical considerations.

(b) A reduction in the roof level diaphragm deflection component, by increasing the diaphragm "effective stiffness".

The effect of increasing or decreasing the average transverse tie stiffness in lateral bending is clearly shown in Fig. 6.4.2(b). Use of very flexible lateral ties could result in quite large dynamic load magnification effects. On the other hand, the use of stiff lateral ties would reduce differential deflections with consequently improved rack performance. Other measures could be taken to increase the diaphragm stiffness which might include, for example, additional bracing and/or use of framing action by rigidly connecting longitudinal and transverse members at node points.

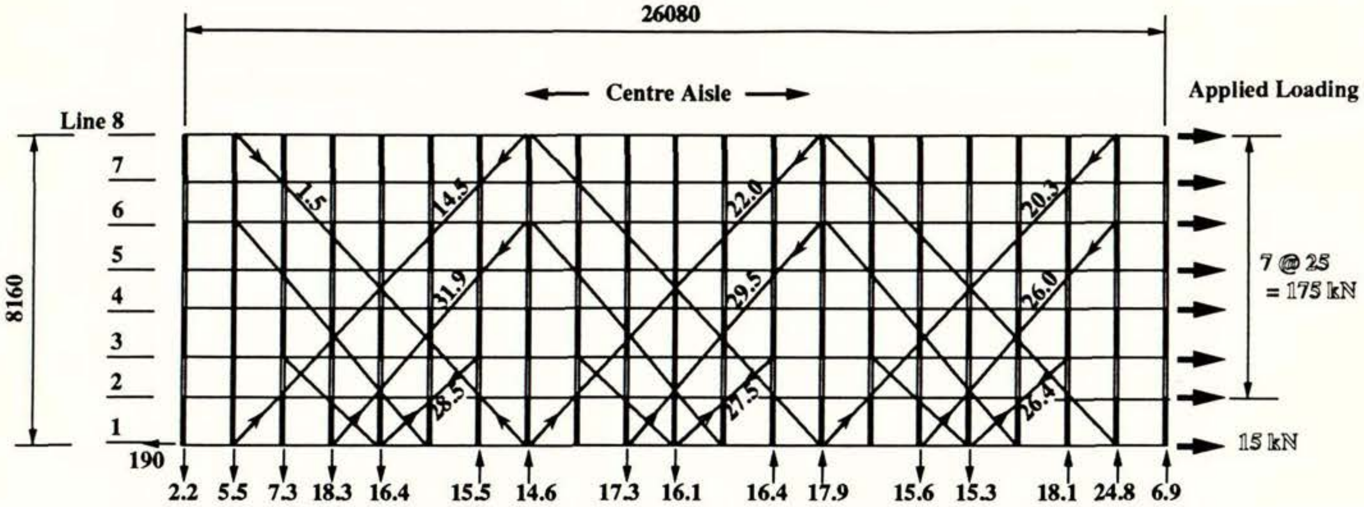


Plan View

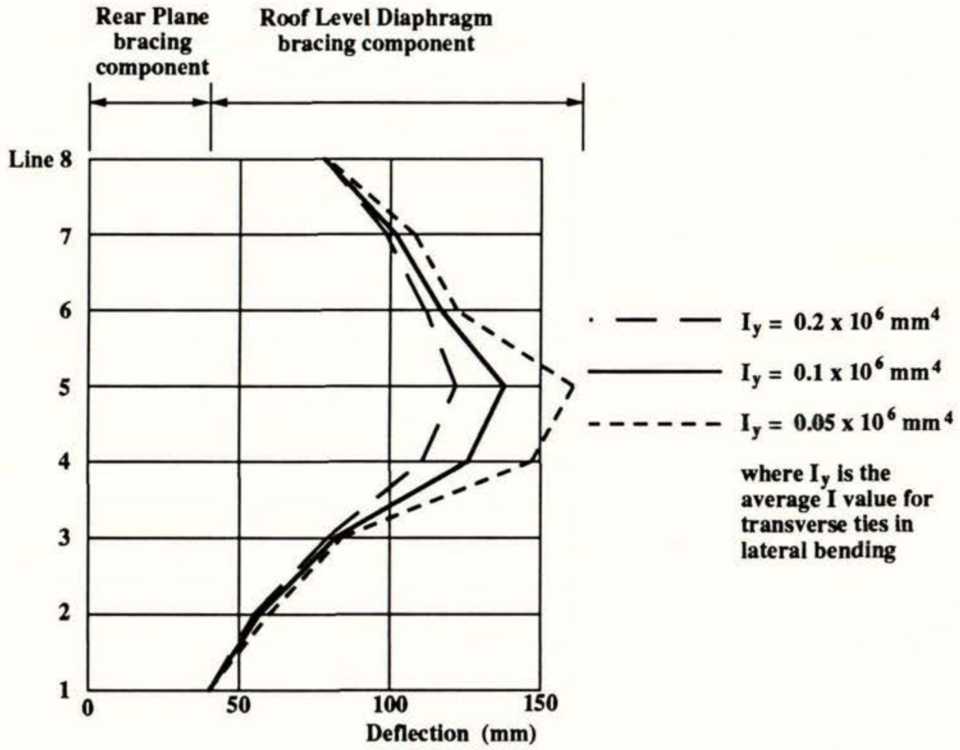
Member Properties

	A (mm^2)	I_x (mm^4)	I_y (mm^4)	J (mm^4)
BTIE	900	0.5×10^6	1.0×10^6	0.001×10^6
LTIE	500	0.945×10^6	0.01×10^6	0.001×10^6
TTIE	300	0.15×10^6	0.1×10^6	0.001×10^6
WIRE	38.5	0.001×10^6	0.001×10^6	0.001×10^6

Fig. 6.4.1 Roof Level Diaphragm Computer Model, TDI



(a) Plan on Diaphragm showing Applied Loading, Bracing Loads and Reactions (kN)



(b) Plan Deflected Shapes

Fig. 6.4.2 Output Data For Model TDI (Roof Level Diaphragm)

6.5 RACK MODEL:

6.5.1 Model Geometry:

To investigate the interaction of the COL10 model with other components of the rack structure, a grillage model, named RACK10, was prepared. This is detailed in Fig. 6.5.1 and models column members, pallet rails, back tie members and the roof level diaphragm (incorporating bracing elements).

The model is effectively an interior line of upright columns loaded in the longitudinal direction. The frame action contribution from top and back tie members, and partial base fixity effects, were modelled as rotational springs calculated as described in Appendix 3. The roof level diaphragm is modelled as a series of springs, using output from the TDI model described in Section 6.4.

6.5.2 Model Analysis:

The following analyses were carried out:-

(a) Eigen Value;
(b) Equivalent Static Analysis (ESA);
(c) Response Spectrum Analysis (RSA).

For the ESA, horizontal forces were distributed in accordance with Clause 3.4.6 of NZS4203 [4]. A series of analysis runs were completed, with varying restraint conditions and to investigate the effect of pallet rail stiffness and top and back tie framing action.

6.5.3 Results:

A summary of output from the RACK10 analysis is presented on Figs. 6.5.2 and 6.5.3. To assist comparison of output data from the ESA and RSA analyses, the RSA data has been scaled, such that the sum of the top (Level 4) spring forces and column base shears is equal to the design earthquake force used in the ESA.

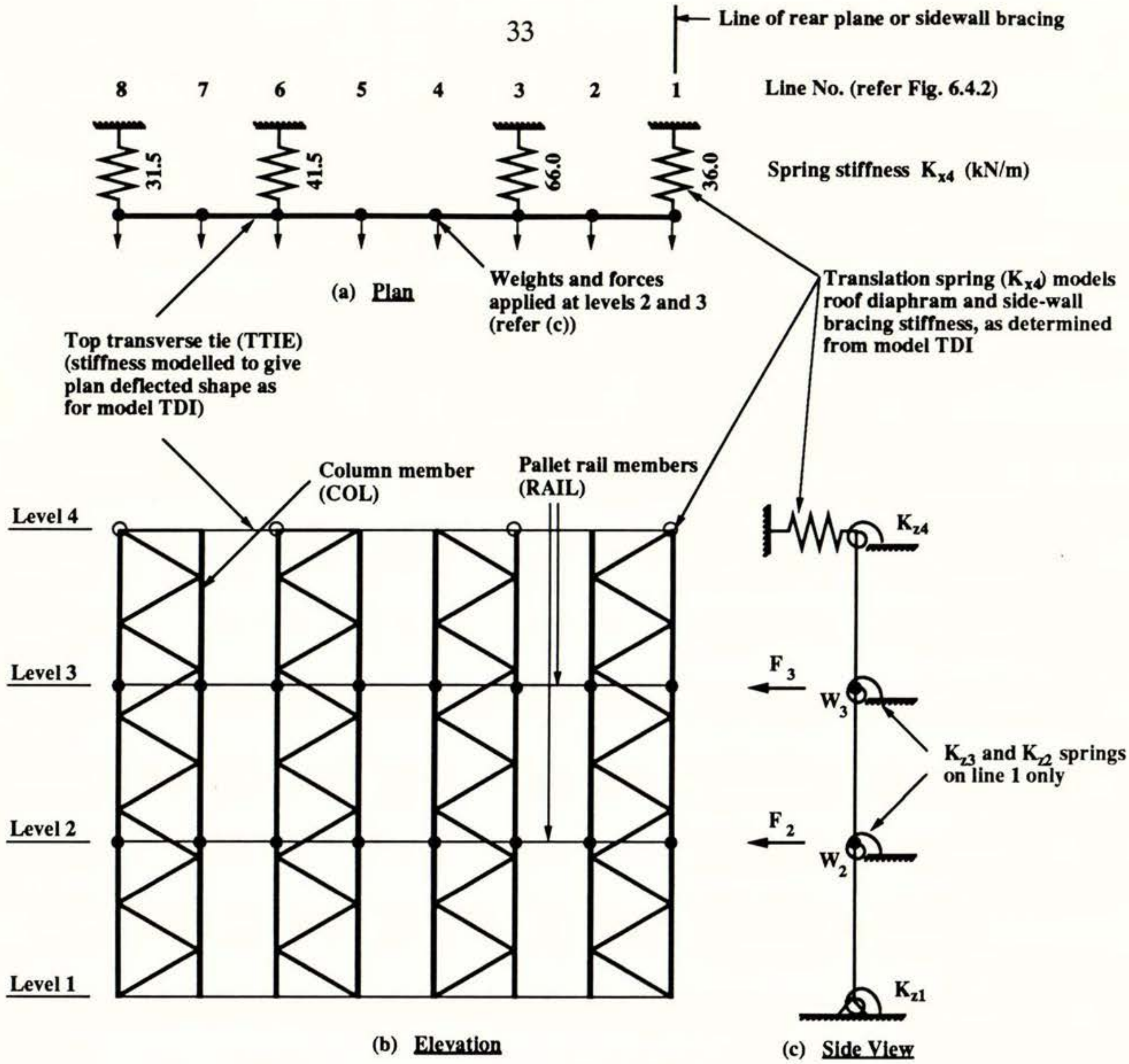
Fig. 6.5.2 presents data for 2 cases, to illustrate the possible influence of pallet rails (depending on form and connectivity) on the structural response of a racking system. The effect of pallet rails on column and brace loads is not normally considered in design. However, a comparison of the deflected shapes plotted in Figs. 6.5.2 (a) and (b) clearly indicates that continuous pallet rails may significantly alter structural response and member loadings.

Referring to Fig. 6.5.2 (a), for the case of pallet rails removed from the model, it is apparent that dynamic amplification effects are a maximum in the column on line 5. Data for this column is presented in Fig. 6.5.3, for the case of a column with end rotational restraints as well as the pin-ended column case. In the dynamic analysis, as compared with an equivalent static analysis, deflections are increased by up to about 30%, while shear forces and bending moments show increases in the order of 50%.

In the case of pallet rails included in the model, dynamic amplification effects are a maximum in the column on line 8, and in this particular case this is also the most critically loaded column. An effect of including the pallet rails is to reduce deflections (and hence to reduce P-delta effects). However, column maximum shear forces and moments are not necessarily reduced and may be difficult to predict apart from detailed modelling of the structure.

The effect of including column end rotational restraints is to reduce column deflections and bending moments, as might be expected. For the case shown in Fig. 6.5.3 the column base restraint has most effect and, for example, reduces the column bending moment at level 2 by 26% in the case of the linear dynamic analysis. Brace loads are reduced by about 10%, indicating that frame action in the RACK10 model with semi-rigid connections resists about 10% of the seismic load. This represents a relatively minor reduction in brace loadings, but may be utilised in design, if required.

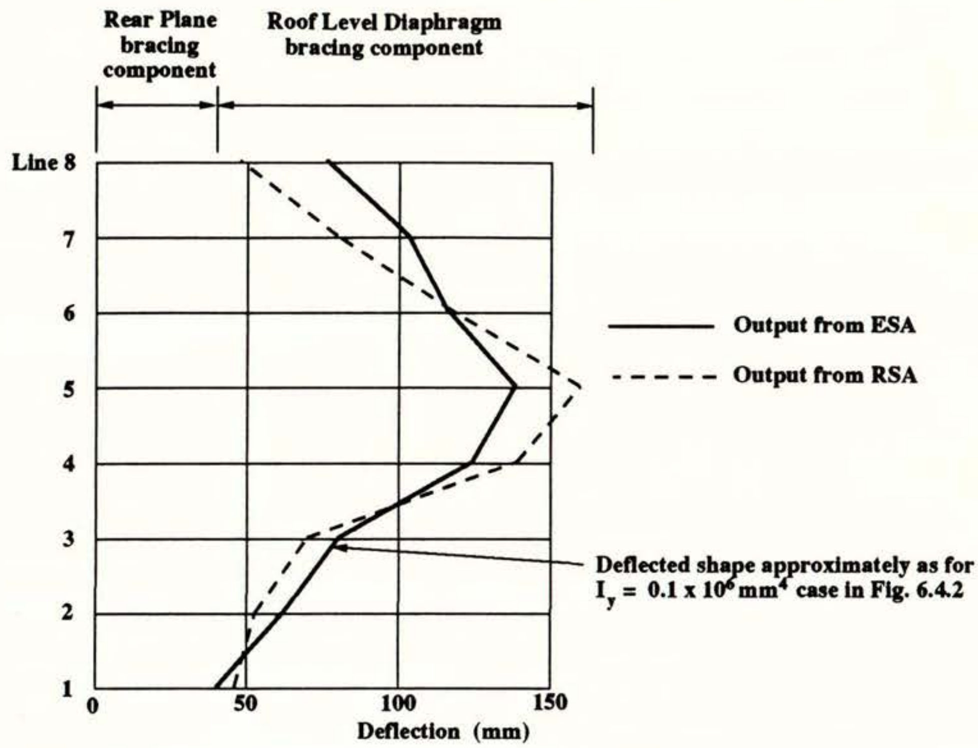
The computed first mode period of vibration of the RACK10 structure with pallet rails removed from the model is 1.72 seconds for columns with pinned ends. This reduces to 1.58 seconds if column end rotational restraints of 50 kN-m/radian and 180 kN-m/radian are included at column bottom and top respectively. These periods reduce about 5% if pallet rails are included in the model.



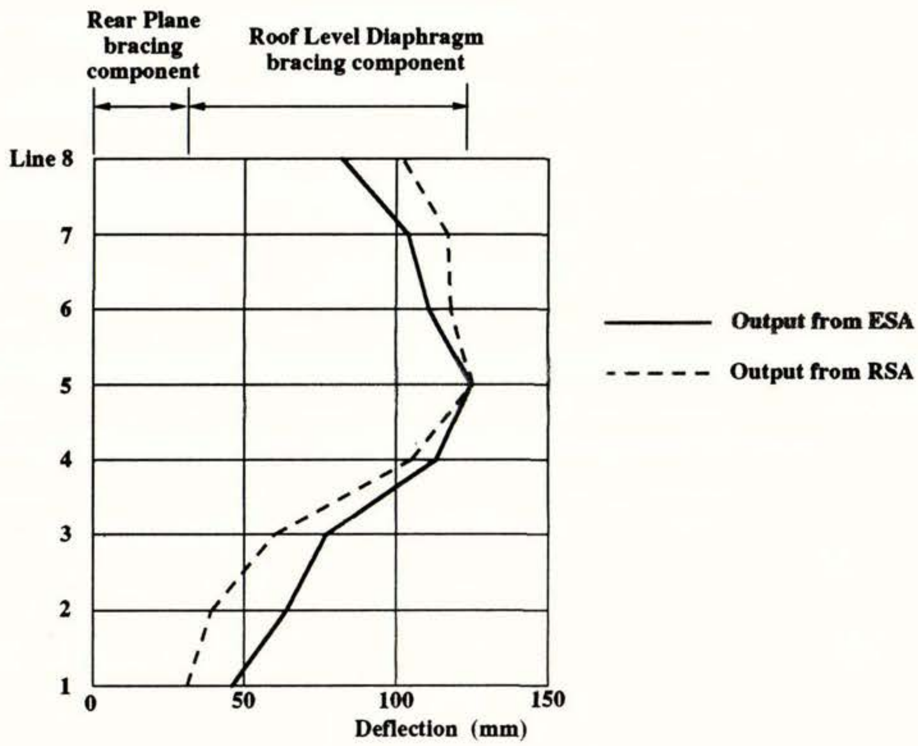
Member Properties

	A (mm^2)	I_x (mm^4)	I_y (mm^4)	J (mm^4)
COL	854	2.08×10^6	0.46×10^6	0.003×10^6
TTIE	300	0.15×10^6	0.17×10^6	0.001×10^6
WIRE	800	1.0×10^6	0.7×10^6	0.001×10^6

Fig. 6.5.1 RACK10 Model Under Longitudinal Earthquake



(a) Plan Deflected Shape for the Case of Pallet Rails Removed from the Model and Columns Pinned Each End



(b) Plan Deflected Shape for the Case of Pallet Rails Included in the Model and Columns Pinned Each End

Fig. 6.5.2 RACK10 Column Top Deflections
(c.f. Fig. 6.4.2)

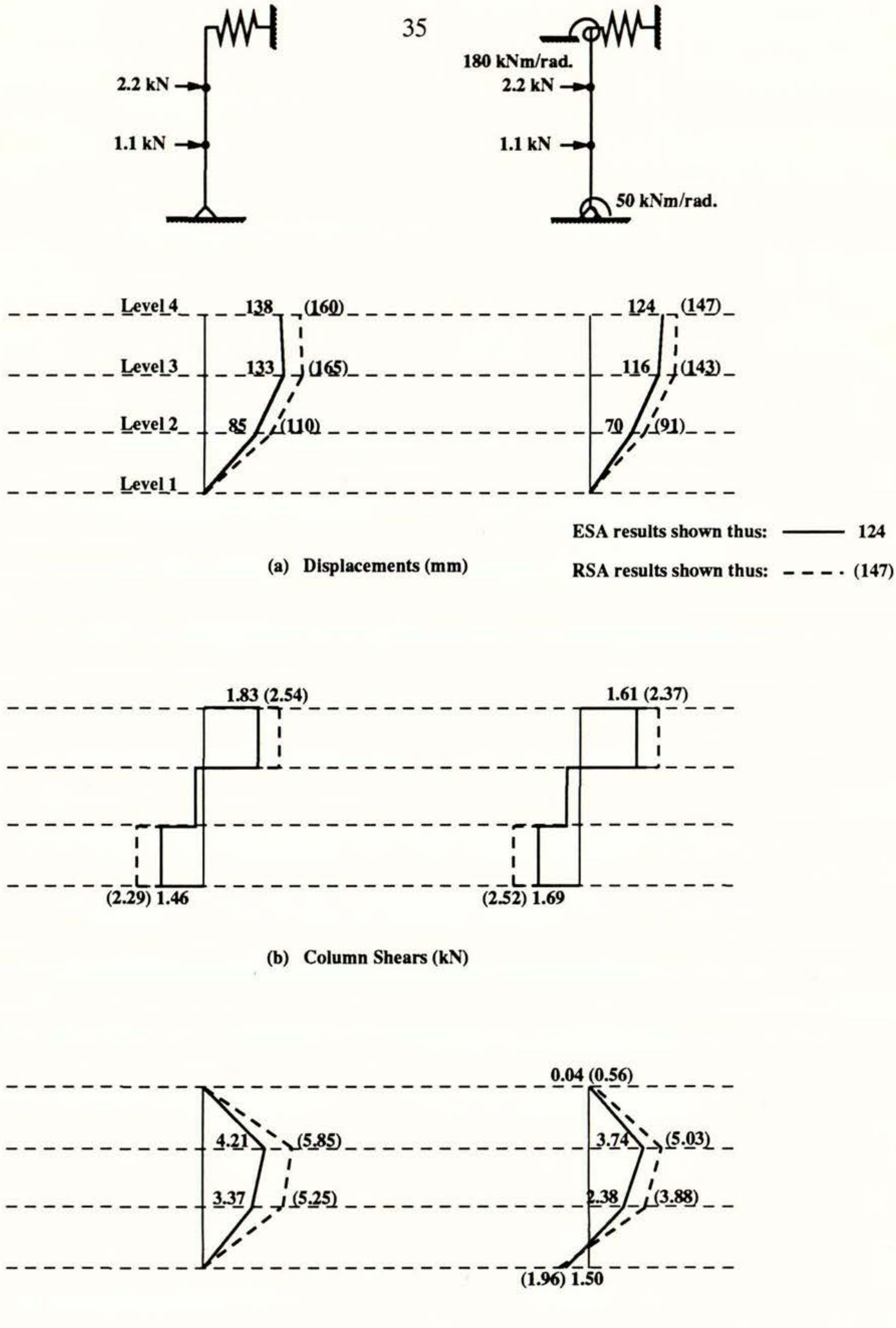


Fig. 6.5.3 RACK10 Output Data For Column On Line 5
(For the Case of Pallet Rails Removed from the Model)

7 DYNAMIC AMPLIFICATION EFFECTS

Dynamic amplification effects may occur due to resonance between components of the rack structure ("in-structure" resonance), or resonance between the rack structure and the supporting building. Such quasi-resonance effects are recognised in Clause 3.6.6.1 of NZS4203 [4] which states:-

> *3.6.6.1 Equipment with a ratio of first mode period to the building design fundamental period in the range 0.6 to 1.4 or equipment mounted on the ground with a first mode period between 0.05 and 2.0 seconds, and the fixings to such equipment, shall be designed to withstand at least twice the force determined using clause 3.4.9.*

In the case of the drive-in rack structure loaded in the longitudinal direction, dynamic amplification effects occur due to interaction between columns, rails, ties and bracing elements. Variations in column forces and deflections arising from different support constraints are amplified in a dynamic loading situation (analogous to the redistribution of forces over the height of a building for an "equivalent static analysis" as prescribed in Clause 3.4.6.1 of NZS4203 [4]).

For the structures investigated in this study, amplification factors of about 1.3 to 1.6 were recorded, indicating a value of 1.5 may be applicable in the design of practical structures, where resonance effects do not occur. An approximate estimate of amplification may be found as the ratio of the peak to average column top deflection under static loading (as determined from a model such as TDI). Accurate determination of the dynamic amplification effects for a specific structure requires a dynamic analysis of a model adequately representing the structure.

Because rack structures are normally only lightly loaded at roof level (i.e. diaphragm self weight only), the onset of resonance, between e.g. columns and the top roof plane diaphragm elements, is unlikely. However, in structures where individual elements (e.g. columns and top bracing elements) do have similar natural frequencies, this must be taken account of in design. In such cases a dynamic amplification factor of at least 2 should be applied to the design column moments.

8 P-DELTA EFFECTS ON UPRIGHT FRAME COLUMNS

8.1 GENERAL:

P-delta effects are the increased moments, shears and deflections caused by the gravity load being laterally displaced in the deformed structure due to seismic or wind forces or other effects.

The flexible form of many rack types in the longitudinal direction, and, in particular, the flexibility of the upright columns in bending, means that a P-delta analysis will often be required. The deformation limits specified in NZS4203 [4] above which P-delta effects must be considered are:-

0.0100H in Zone A
0.0083H in Zone B
0.0067H in Zone C

These limits are applied to the computed deformations resulting from the design earthquake forces specified in Clause 3.2 of DG 8.3 [3] multiplied by the factor $K^{l}(0.7\ SM)$, where K is as defined in Clause 3.8.1 of NZS4203 [4].

8.2 COLUMN MOMENT AMPLIFICATION:

Column moment amplification due to P-delta effects may be computed as follows:-

(a) Elastically Responding Structures:

(i) Calculate column moments and deflected shape using the equivalent static force analysis method. If a non-linear analysis computer programme is used, this will output the column moments incorporating P-delta effects and no further analysis is required. If linear analysis only is employed, proceed to step (ii);

(ii) Calculate reactions induced at each end of the column due to the initial deflection of gravity loads, and hence calculate the net moment induced at each pallet level;

(iii) Define the stability coefficient, as:

$$\theta_m = M_{ix}/M_m$$

(iv) Calculate the moment amplification factor, as:

$$K_{mx} = 1/(1- \theta_m)$$

where M_{ix} = the induced moment at level x due to the initial deflected shape.

M_m = the bending moment at mid height of the column from linear analysis.

K_{mx} = the moment amplification factor at level x.

Typical values for K_{mx} would be in the order of 1.5 (refer Fig. 8.4.1), although values of up to about 2.0 may apply for columns with high earthquake induced axial loadings (refer example in Appendix 5).

(b) Yielding Structure:

A drive-in rack structure with longitudinal bracing capable of plastic deformation, or incorporating an energy absorbing element, may give an increased column top deflection over that assuming elastic response only of bracing.

The response of this type of structure to earthquake loading is best predicted using an inelastic time history analysis programme. However, for the purpose of a P-delta analysis, column deflection may be approximated by setting up an elastic model as follows:-

(i) Load reduction is taken account of in the SM factor from Appendix 1, with allowance for overall damping (including 1-2% allowance for rocking in the palletised loads);

(ii) Brace stiffness is adjusted to model the secant stiffness of the bracing element. A suitably modified brace stiffness may be calculated from:-

$$K^1 = K \times SM/\gamma$$

The factor γ is chosen depending on the damping available in the structure framework. Values range from 2.0 for systems with 8% damping (typically numerous semi-rigid joints and large frame deformations of, say, 0.04 H) to 4.0 for systems with 3% damping (typically low frame deformations of, say, 0.01 H).

The "equivalent" elastic model as above is then analysed for P-delta effects as for an elastically responding structure (refer paragraph (a) above).

It is assumed that yielding of the upright frame column member does not occur. Note that if plastic hinges did form in the columns this could lead to the development of a "soft storey" and progressive collapse throughout the rack structure.

8.3 P-DELTA AMPLIFICATION OF BRACE LOADINGS:

Brace load amplification due to P-delta effects may be calculated in the same way as column moment amplification (refer Section 8.2) with steps (ii) and (iii) of 8.2(a) modified as follows:-

(ii) Calculate reactions induced at each end of the column due to the initial deflection of gravity loads.

(iii) Define the stability coefficient, as:

$$\theta_c = R_i/R$$

(iv) Calculate the column top reaction amplification factor, as:

$$K_c = 1/(1-\theta_c)$$

where R_i = the induced reaction at the column top due to the initial deflected shape

R = the reaction at the column top from a linear analysis

K_c = the column top reaction amplification factor.

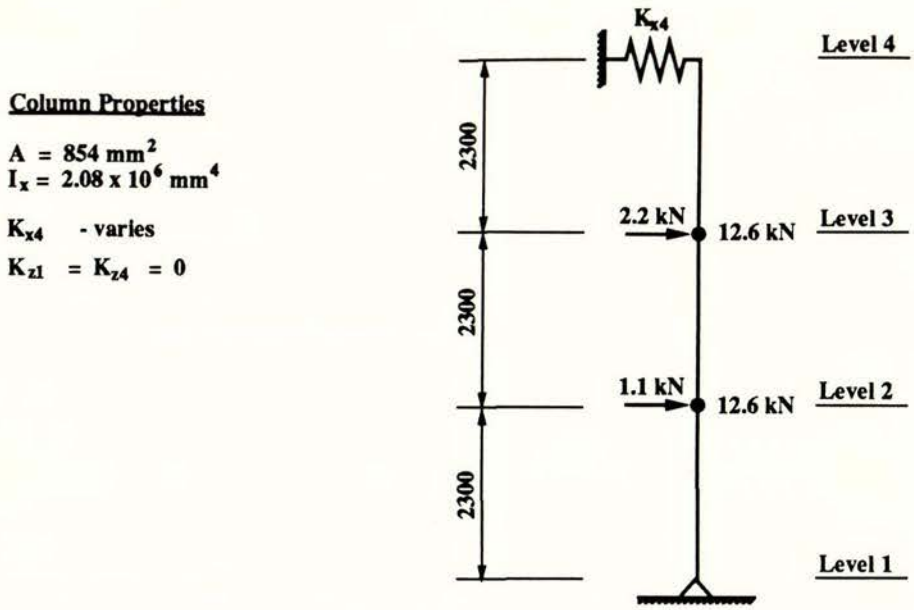
Note that the brace load amplification factor, K_b, may be taken equal to K_c provided that K_c is the average value for all columns supported by the brace under consideration. Typical values for K_c would be in the order of 1.3 (refer Fig. 8.4.1), although significantly higher values can apply in the case of columns with high earthquake induced axial loadings (refer example in Appendix 5).

8.4 COLUMN P-DELTA AMPLIFICATION FACTORS FOR A TYPICAL PALLET RACK COLUMN:

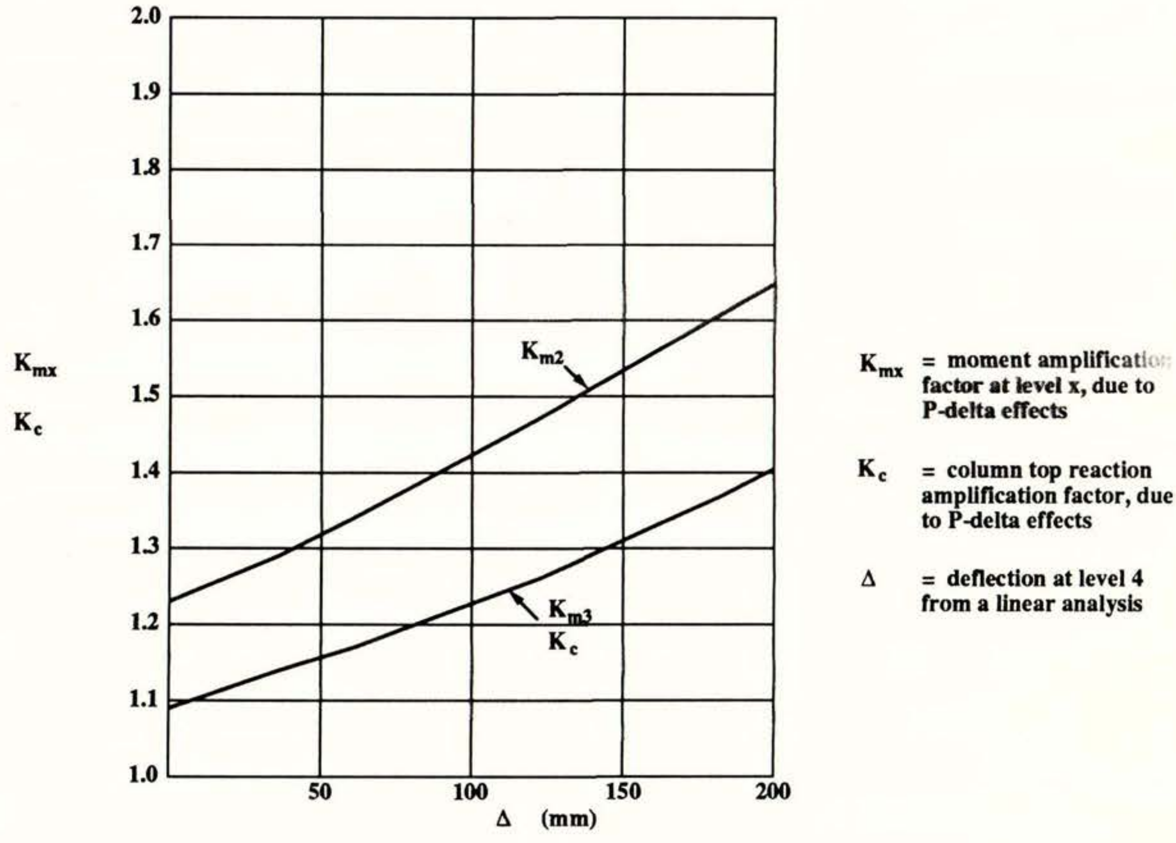
For a typical installation as shown in Fig. 8.4.1(a), the column moment amplification factor due to the P-delta effect has been plotted against column top displacement in Fig. 8.4.1(b). This illustrates the order of increase in column moment associated with the use of flexible bracing.

For example, increasing the displacement at the top of the column from 0 to 200mm would result in about a 34% increase in column moment at level 2, for this particular case.

Even for zero column top displacements (infinitely stiff bracing), it is apparent that the P-delta effect is significant and must be allowed for in design. This is not unexpected, as the inter-storey deformation between levels 1 and 2 is about twice the limit above which P-delta effects must be considered. The maximum moment amplification occurs at level 2 (the lower pallet rail support level), and this will often be the critically loaded part of the column.



(a) Column Model for P-Delta Amplification Analysis



(b) Plot of Amplification Factors vs Column Top Deflection

Fig. 8.4.1 Column P-delta Amplification Factors

9 HOST BUILDING REQUIREMENTS AND TYPICAL DETAILS

9.1 GENERAL:

The HERA Design Guide DG 8.3 [3] details specific requirements for enclosing buildings. These include:-

(a) Where lateral resistance to earthquake loading is provided to the storage rack by the superstructure of the enclosing building, the adequacy of the building to resist such forces shall be confirmed by the specifier.

(b) Adequate separation shall be maintained between the rack structure, and the fabric of the enclosing building, to permit predicted earthquake deformations to occur.

(c) Floor systems in buildings enclosing storage racks shall be of adequate strength to support loads (including earthquake induced loads) imposed by the rack.

(d) Ancillary equipment, e.g. fire sprinkler pipework, attached to the rack structure shall be of suitable design and location, so as not to influence rack behaviour in an earthquake.

Refer also to the Schedule in Appendix 6, reproduced from Appendix D of DG 8.3[3], which includes a section on structural aspects of the host building.

9.2 TYPICAL DETAILS:

A selection of typical host building details is presented in Figs 9.2.1 to 9.2.5. These are briefly discussed in the following sections 9.2.1 to 9.2.3.

9.2.1 Foundation and Floor Details:

For a typical warehouse with rack storage such as that detailed in Fig 9.2.1, provision for rack loadings may be incorporated relatively simply into the floor construction details. The general concepts proposed are illustrated in Fig 9.2.2. These include:-

(a) A minimum recommended floor strength and thickness is indicated . Where rack design loads are known in advance, the floor details may be selected to suit anchorage loadings. For example, the required load capacity will determine the minimum anchorage embedment required for a given concrete strength, to protect against a brittle concrete cone type failure of the anchorage.

(b) Ground slab cast integral with foundation beams, to mobilise maximum available structure weight to resist uplift forces.

(c) Increased slab thickness provided under front and rear faces of the rack structure to provide "strong bands" at these locations, with good load redistribution capabilities and provision for adequate anchor embedment depth.

9.2.2 Rack Hold-Down Details:

Alternative details for rack hold-down are presented in Figs 9.2.3 and 9.2.4. Anchor size should normally be M12 or larger.

The use of a floor channel as shown in Fig 9.2.3 assists in load redistribution between columns as well as providing "in effect" a small kerb affording some protection against forklift impact damage to columns. Alternative 2, detailed in Fig 9.2.4, incorporates a thin baseplate thereby introducing some flexural yielding capability into the system.

9.2.3 Post-Installed Anchors:

In an existing building where the foundation and/or floor capacity is inadequate for the imposed rack loadings, post-installed mini-piles or ground anchors as shown in Fig 9.2.5 may be used to increase the available anchorage capacity. In the case of an insulated building such as a cool store, particular attention to detail would be required to maintain the insulation barrier. The safe working load of this type of anchor depends on anchor details and site conditions, but typically would be in the range of 20 to 50 kN.

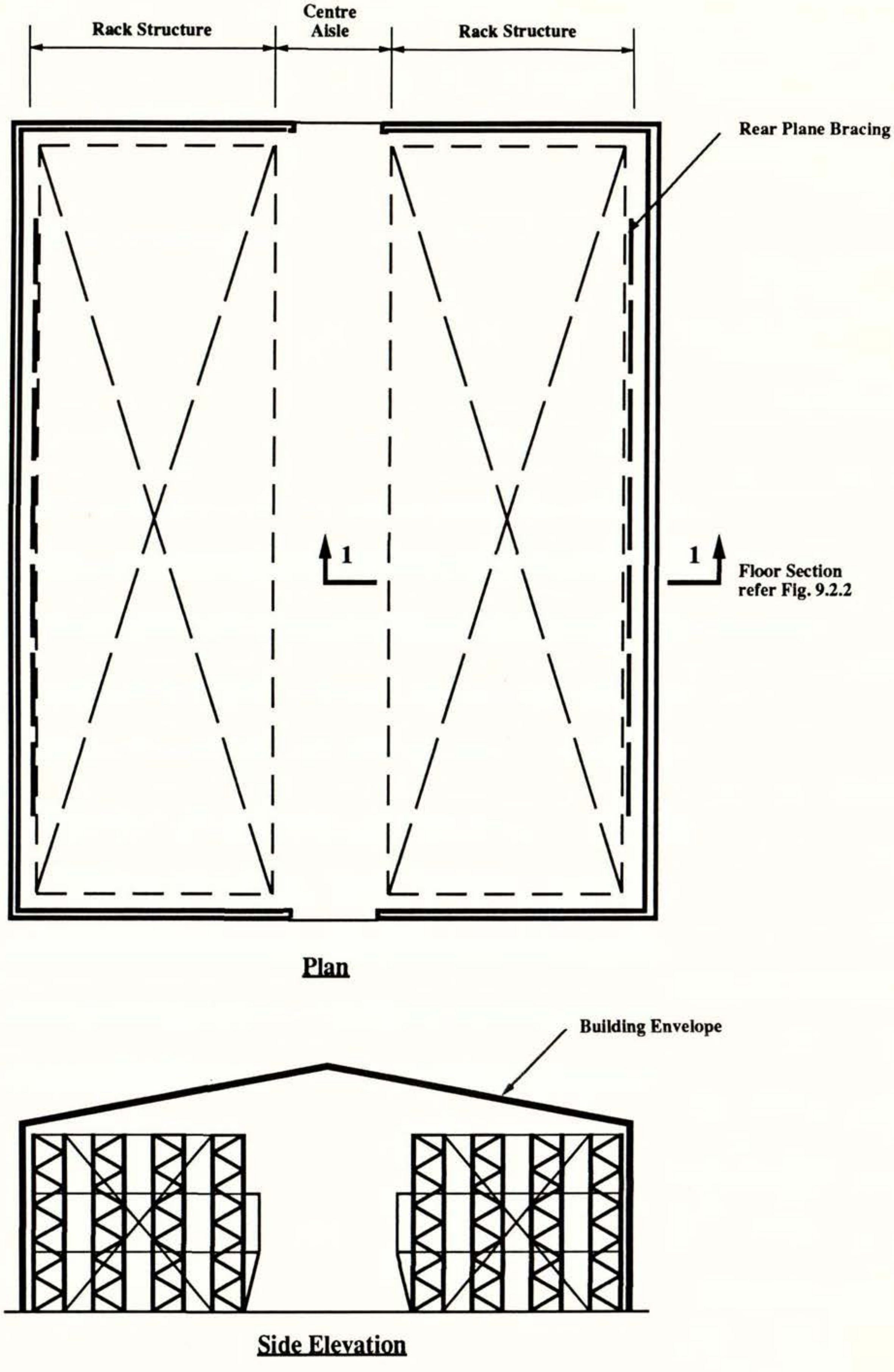
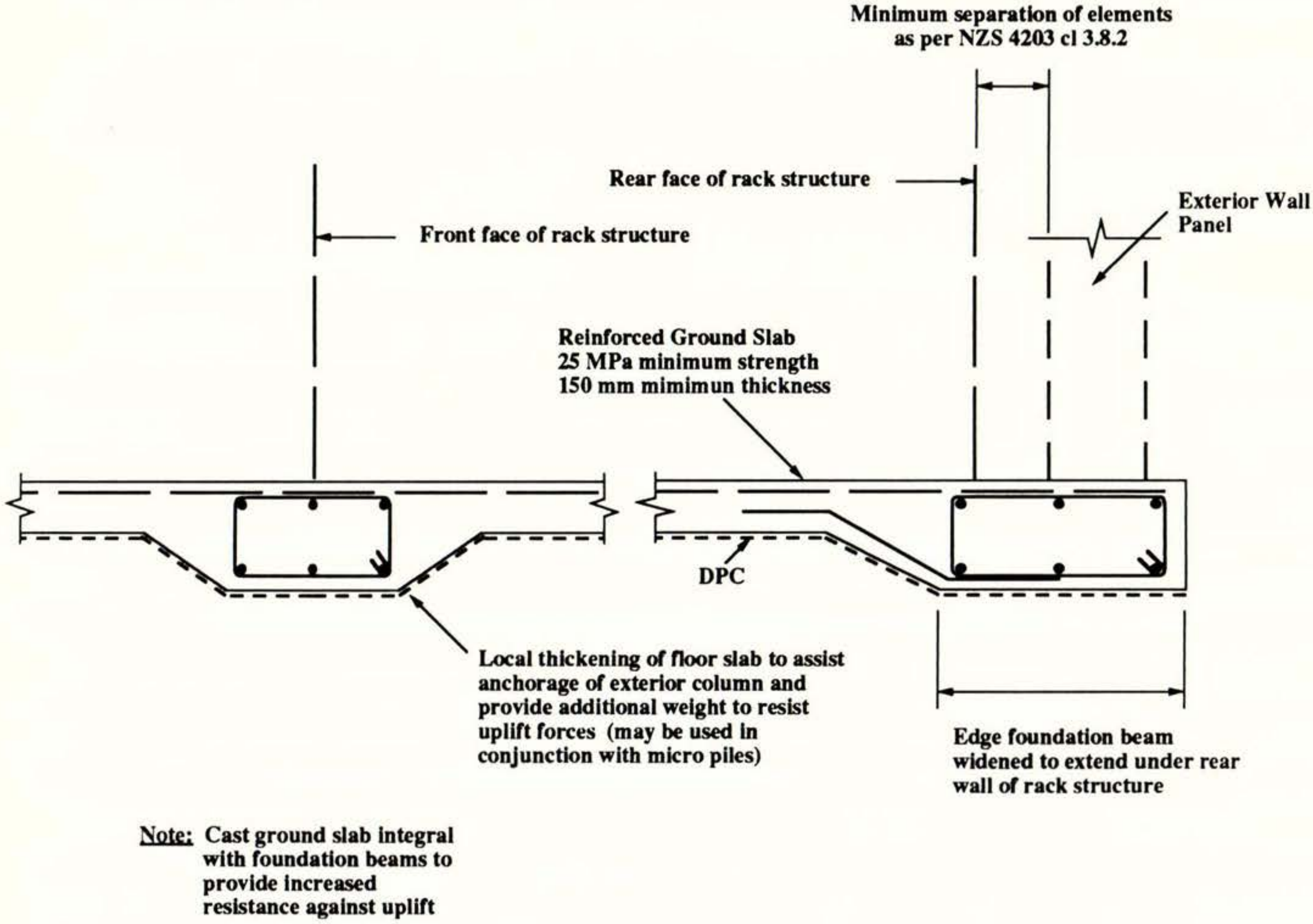
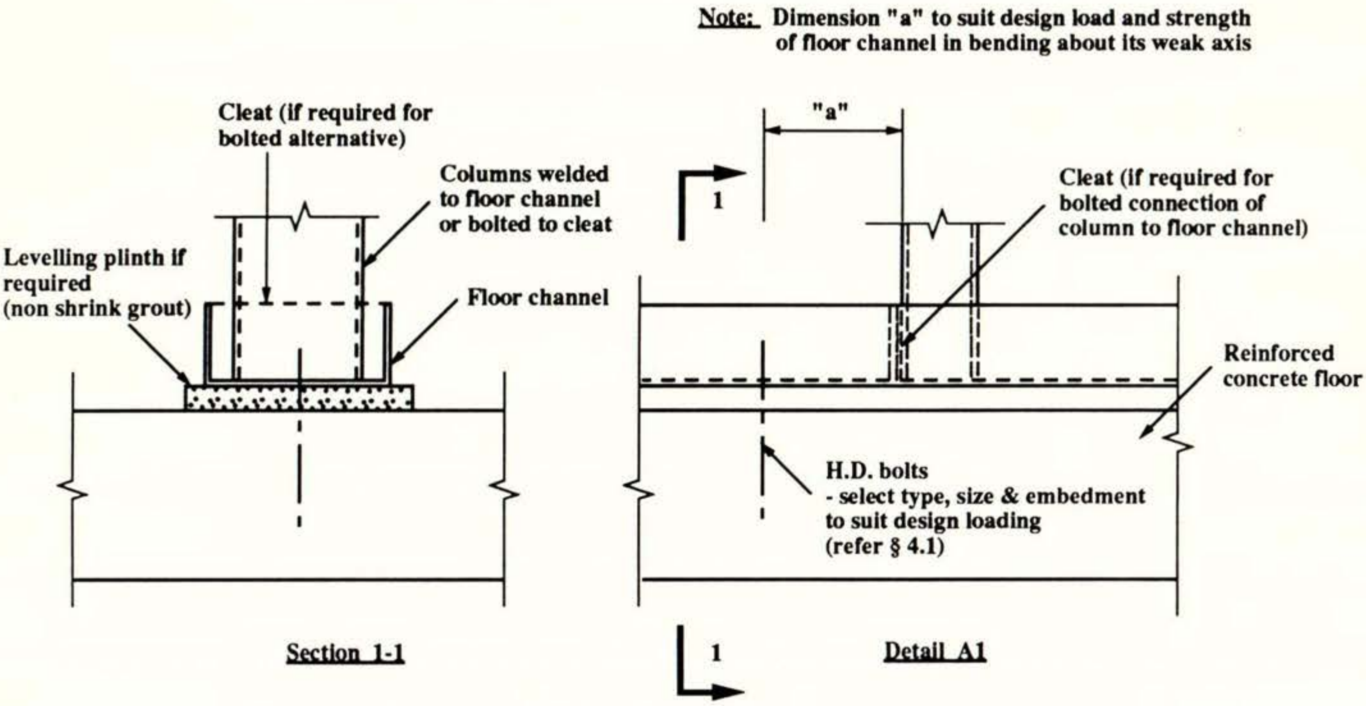


Fig. 9.2.1 Typical Warehouse With Rack Storage



Floor Section 1-1
(refer Fig. 9.2.1)

Fig. 9.2.2 Warehouse Floor Details



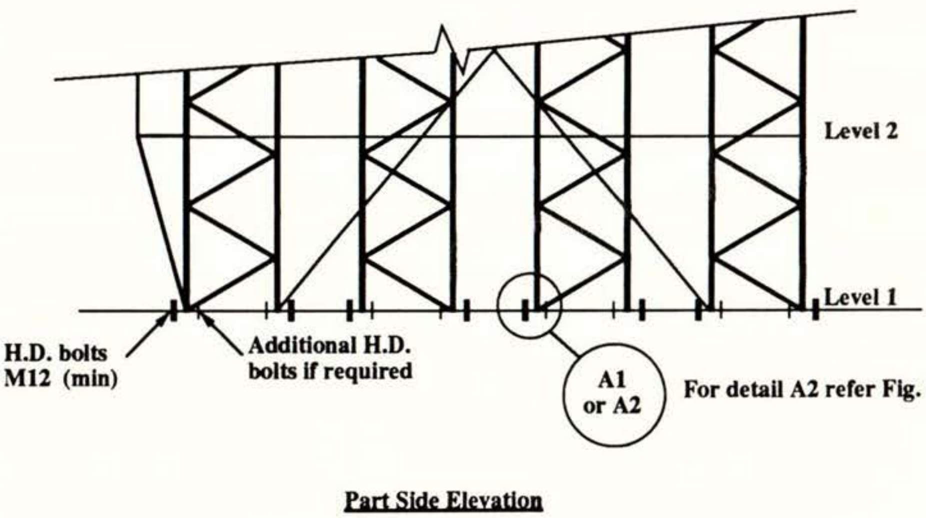


Fig. 9.2.3 Rack Hold-Down Details - Alternative 1
(Column Fixed To Floor Channel)

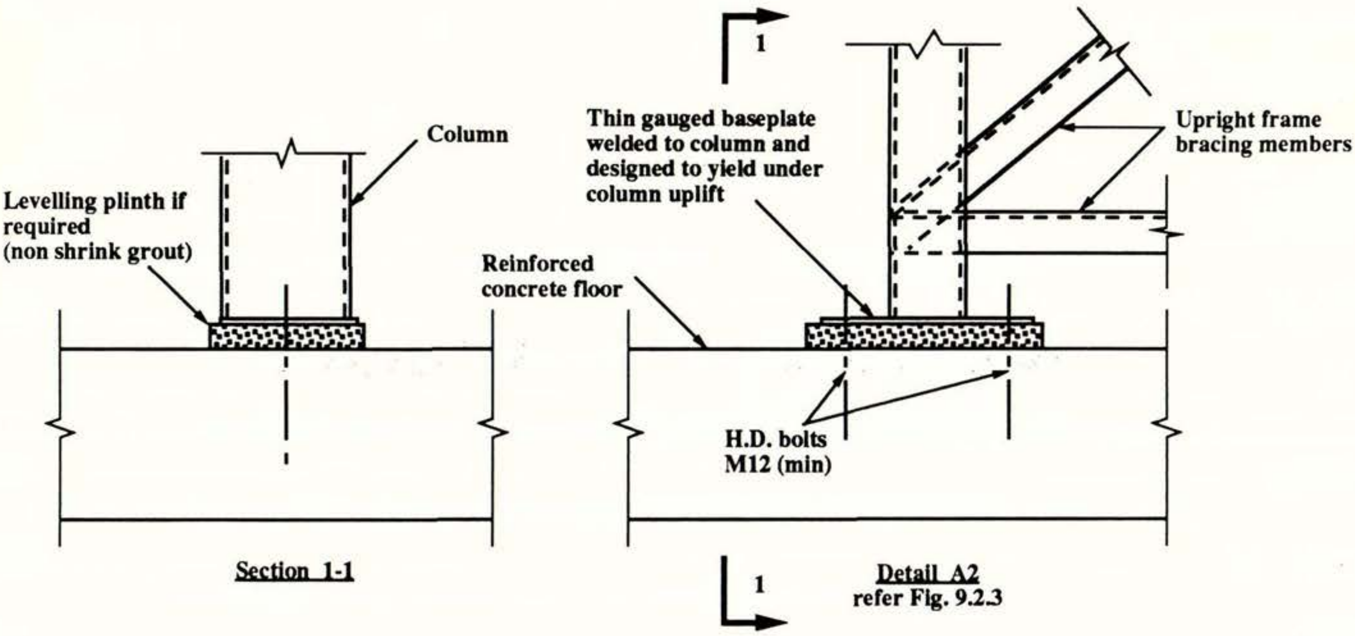


Fig. 9.2.4 Rack Hold-Down Details - Alternative 2
(Column Fixed To Flexible Baseplate -
May Be Used With Or Without Floor Channel)

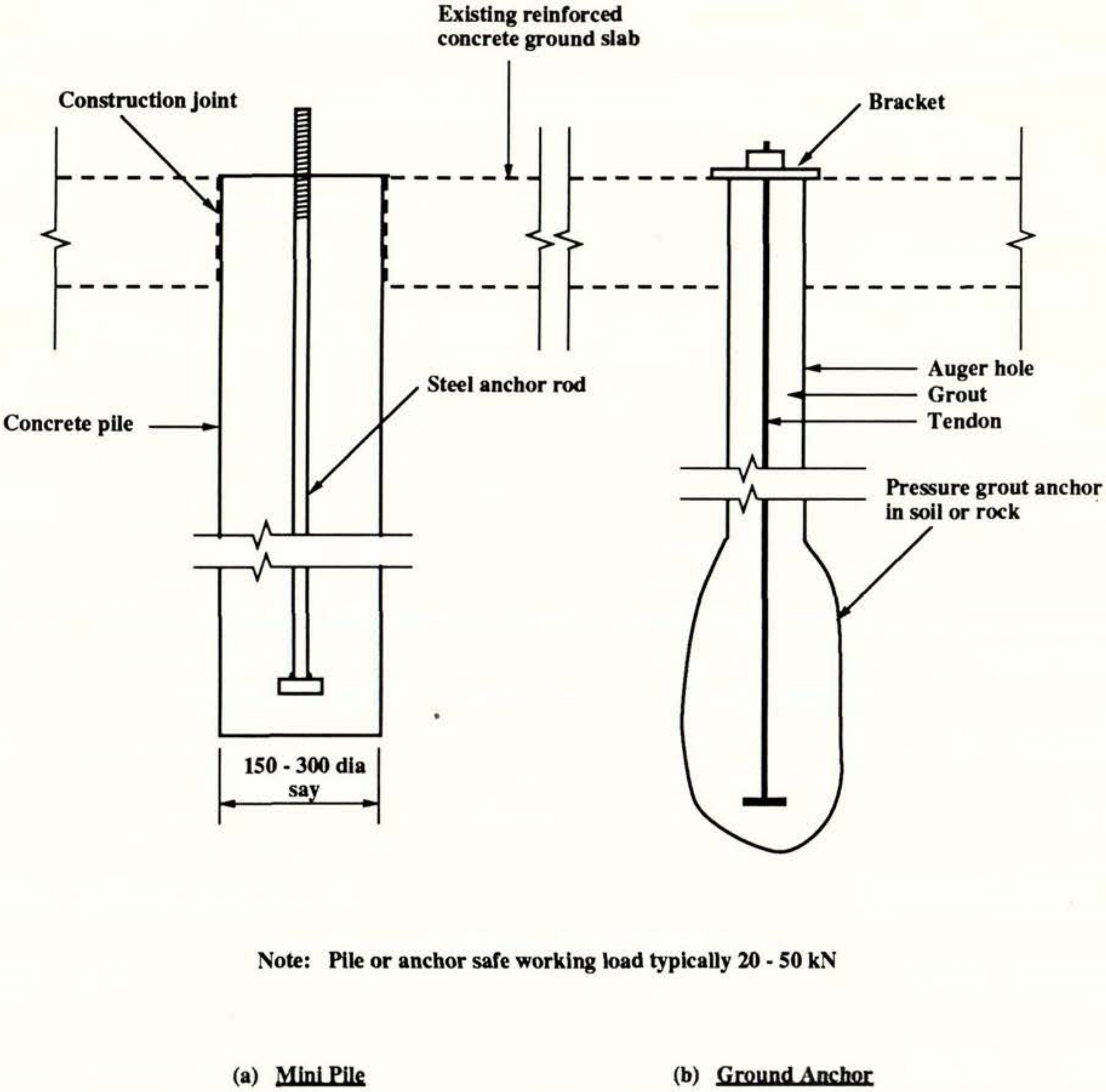


Fig. 9.2.5 Post-Installed Anchors For High Local Loading
(Eg. Bracing Wire Anchorage)

10 SUMMARY AND RECOMMENDATIONS

The results of this study, and recommendations arising from the work carried out, are summarised in the following Sections 10.1 - 10.8. The approach recommended is to consider the component parts, and how they influence the response, of a typical drive-in rack structure.

10.1 BRACING CONSIDERATIONS FOR EARTHQUAKE LOADING IN THE LONGITUDINAL DIRECTION

- The response of column elements to seismic loading is strongly dependent on the range and variability of the column top deflections, which comprise a roof level diaphragm and a rear plane bracing deflection component. Increased deviations of the deflected shape from the average deflection correspond to increased dynamic amplification of loadings in elements at those locations. Thus, improved rack performance would result from:-

  (a) Providing for an increase in the rear plane bracing proportion of total deflection, with a maximum deflection limited by stability and practical considerations.

  (b) A reduction in the roof level diaphragm deflection component, by increasing the diaphragm "effective stiffness".

10.2 REAR PLANE OR SIDE-WALL BRACING

- Rear plane or side-wall bracing provides support to the structure under longitudinal earthquake loading. A flexible bracing system is recommended, as allowing for increased structure deflection under seismic loading has the following advantages:-

  (a) Structure response to seismic loading is reduced due to increase in structure period.

  (b) Structural damping generally increases with increasing deflections.

- Tensioned wire bracing is an effective rear plane bracing system. Bracing stiffness increases with wire pretension, so the level of wire pretension in the rear plane bracing should normally be minimal. Note, however, that wires should be evenly tensioned to ensure even distribution of loading. A minimum factor of safety of 2 against wire breakage should apply.

- While the use of anchor frames in conjunction with longitudinal bracing has not been explored in this study, it is believed that this would be beneficial in many cases, as deformations in the semi-rigid joints of the anchor frames would increase the overall system damping.

10.3 ROOF LEVEL DIAPHRAGM

- It is recommended that, where possible, the roof level diaphragm be continuous across the centre aisle and span between rear plane or side-wall bracing lines, to minimise eccentric seismic loading effects. The use of symmetrical seismic load resisting systems is a fundamental principle of design for structural resistance to seismic loadings.

- In situations where the roof level diaphragm is supported in the longitudinal direction along one edge only, the upright frames acting in conjunction with supplementary bracing can be detailed to provide the required torsional resistance. This case is developed in the model analyses and design examples contained in this report. However, as illustrated in the design example in Appendix 5, torsional loadings can have a significant effect on column sizing. This is because quite large additional axial loads may be induced in columns which are already laterally loaded and deflected in the longitudinal direction as a result of the direct component of earthquake loading.

- In general, it would appear that structure performance is likely to improve with increasing diaphragm stiffness. This is because:-

  (a) In the longitudinal direction, it is desirable to control column top deflections to be close to the average deflection. This will minimise dynamic amplification effects and ensure that, as far as possible, columns are evenly loaded.

  (b) In the transverse direction, loading of upright frame lines will be more evenly distributed as diaphragm stiffness increases.

10.4 UPRIGHT FRAMES

- To ensure satisfactory frame performance under lateral seismic loading, careful attention should be given to design and detailing. Experimental verification of the design is required, unless the design and detailing complies with the requirements of an approved "means of compliance" document. Tests on "standard" frames not specifically detailed for seismic loading have indicated relatively poor seismic performance.

- Design of upright frame column members in strong axis bending under longitudinal seismic (face) loading must take account of P-delta and dynamic amplification effects, as well as any earthquake induced axial loads. Load carrying capacity is increased by detailing for end restraints to provide at least partial end fixity.

- Bracing details must be adequate to ensure proper bracing of column members in flexural and flexural-torsional buckling considerations. Bolted connections must be fully tightened for end restraints and bracing connections to be fully effective.

- Column base anchorage should be designed to yield, and overstrength provided against brittle failure mechanisms. In some cases the base anchorage may be detailed to allow frames to rock under seismic loading. However, it is considered that "rocking frames" would be unsuitable in situations where frames must provide torsional stability to the structure (as in a drive-in rack structure where the roof level diaphragm is not structurally connected across the centre aisle). In this application column longitudinal deflections are affected by the frame transverse deflections, and may be increased excessively if large transverse deflections occur.

- Columns may need to be checked for possible "hydraulic" loading, which may occur due to product spillage under earthquake loadings. Depending on the nature of product and method of storage, this effect can be severe. Measures for reducing possible "hydraulic" loading effects should be considered.

10.5 DYNAMIC EFFECTS

- Typically, dynamic amplification effects only need to be considered in design of columns and those members directly supporting columns with greater than the average deflection. Loadings should be determined from an appropriate dynamic analysis or, alternatively, a dynamic amplification factor of 1.5 should be applied to the results of an equivalent static analysis.

- Pronounced resonance effects are not expected in a typical ground mounted drive-in rack structure, due to the low concentration of mass at roof level. However, in the case of a rack structure which is not ground mounted and/or there is mass concentrated at the roof diaphragm level, the possibility of resonance effects should be investigated. For a ratio of rack structure to building (or column to diaphragm) fundamental period in the range of 0.6 to 1.4, column moments and forces may be amplified by a factor of 2 or more.

10.6 P-DELTA ACTIONS

- P-delta actions have a significant effect in the design of drive-in rack structures and are not adequately provided for by reliance on the provisions of code interaction design formulae.

- P-delta actions may typically amplify column moments by about 50 to 100%, and column top reactions by about 30 to 60%. The higher amplifications would normally apply to columns with high earthquake induced axial loadings adding to gravity loads.

- The deformation limits above which P-delta actions must be considered are:-

  0.0100H in Zone A
  0.0083H in Zone B
  0.0067H in Zone C

10.7 HOST BUILDING REQUIREMENTS

- Where lateral resistance to earthquake loading is provided to the storage rack by the superstructure of the enclosing building, the adequacy of the building to resist such forces should be confirmed by the specifier.

- Adequate separation should be maintained between the rack structure, and the fabric of the enclosing building, to permit predicted earthquake deformations to occur.

- Floor systems in buildings enclosing storage racks should be of adequate strength to support loads (including earthquake induced loads) imposed by the rack.

- Ancillary equipment, e.g. fire sprinkler pipework, attached to the rack structure should be of suitable design and location, so as not to influence rack behaviour in an earthquake.

10.8 "RISK" AND ECONOMIC CONSIDERATIONS

Notwithstanding the mandatory code provisions applying to pallet rack structures (including the effects summarised in Section 10.4), the insurance industry needs to justify economically the necessity for increased standards of seismic resistance above those presently adopted by the industry. Relevant factors here include:-

(i) Risk to life (as opposed to property);

(ii) Relative cost of palletised goods, and storage rack;

(iii) Difficulty in preventing "spillage" of palletised goods in earthquakes, regardless of seismic design standards;

(iv) Temporary nature of the rack structure (as a component in a materials handling system);

(v) Damage from other sources e.g. forklifts;

(vi) Economics.

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12 NOTATION

a	=	distance
A	=	the cross-sectional area of a steel section
A_b	=	the cross-sectional area of a mild steel tension brace
b	=	distance <u>or</u> the column dimension normal to the axis about which bending occurs
B	=	the rack dimension perpendicular to the horizontal loading direction under consideration
C	=	the basic seismic coefficient
C_d	=	the seismic design coefficient
d	=	the column dimension parallel to the axis about which bending occurs
D	=	dead loads
E	=	earthquake loads <u>or</u> the modulus of elasticity for steel
E_c	=	the modulus of elasticity for concrete
f_a	=	the calculated axial stress in a member
f_{bx}	=	the calculated maximum tensile or compressive stress in a member bent about the x-axis
F_a	=	the maximum permissible average compression stress in an axially loaded member not subjected to bending.
F_b	=	the maximum permissible stress in tension and compression on the extreme fibres of a laterally unbraced beam.
F_{bx}	=	the value of F_b for bending about the x-axis
F_{ob}	=	the elastic flexural-torsional buckling stress due to moment alone in a member subjected to combined bending and compression
F_{oc}	=	the elastic buckling stress in an axially loaded compression member
F_{ox}, F_{oy}, F_{oz}	=	the values of F_{oc} for flexural buckling about the x- and y-axes, or torsional buckling

F_{oxz}	=	the value of F_{oc} for flexural-torsional buckling about the x- and z-axes
F_x	=	the horizontal force in the direction under consideration that is applied to the level designated as x
F_y	=	the yield stress of the steel
G	=	the shear modulus of elasticity
G_A, G_B	=	stiffness ratios
h_x	=	the height to the level designated as x
H	=	height or inter-storey height
I_b	=	the second moment of area of a beam about the axis normal to the buckling plane
I_c	=	the second moment of area of a column about the axis normal to the buckling plane
I_f	=	the second moment of area of a fictitious floor beam
I_w	=	the warping section constant
I_x, I_y	=	the second moments of area of the full section about the x- and y-axes
J	=	the torsion section constant
K	=	the equivalent translational spring stiffness of a bracing element or elements
K^1	=	a modified spring stiffness to account for structure ductility
K_b	=	the equivalent rotational spring stiffness of beams framing into a column top <u>or</u> the brace load amplification factor to account for P-delta effects
K_c	=	the column top reaction amplification factor to account for P-delta effects
K_d	=	a load amplification factor to account for dynamic effects
K_e	=	a load amplification factor to account for possible eccentricity of load distribution
K_f	=	the column base rotational spring stiffness

K_{mx}	=	the P-delta moment amplification factor at level x
K_T	=	the effective length factor for torsional buckling
K_x	=	the equivalent translational spring stiffness at level x
K_{zx}	=	the equivalent rotational spring stiffness at level x
K_θ	=	the joint rotation stiffness
l	=	the effective length of a member
l_x, l_y, l_z	=	the effective lengths for buckling about the x-, y- and z- axes
L	=	the length, span, or unbraced length of a member <u>or</u> live loads
L_b	=	the span or the unbraced length of a beam
L_c	=	the length or unbraced length of a column
L_T	=	the spacing of exterior columns in the transverse direction
m	=	the distance of the shear centre of a section from mid-plane of the web
m_x	=	the moment applied at level x due to eccentricity of loading
M	=	the structural material factor <u>or</u> an applied bending moment
M_{ax}	=	the amplified bending moment at level x allowing for P-delta effects and dynamic amplification
M_{ex}	=	the bending moment at level x calculated from a linear-elastic analysis
M_{ix}	=	the induced bending moment at level x due to initial deflected shape
M_m	=	the bending moment at mid-height of a column calculated from a linear-elastic analysis
M_{nx}	=	bending moment at level x including P-delta effects
P	=	an applied force
P_a	=	the calculated maximum load capacity of a compression member
P_E	=	the additional earthquake induced load in an exterior column due to pallet centres of mass above rail levels

P_f	=	the lateral force carried by frame action
P_u	=	an applied force, at strength level of loading
P_x	=	the vertical force applied at level x
Q	=	the form factor of a compression member
r_{01}	=	the polar radius of gyration of a cross section about the shear centre
r_x, r_y	=	the radii of gyration of the full section about the x- and y- axes
R	=	the column top reaction calculated from a linear-elastic analysis or the inside radius of a bend
R_i	=	the induced column top reaction due to initial deflected shape
S	=	the structural type factor
t	=	the nominal steel thickness of a section
T	=	the calculated tension force in a brace
T_a	=	the design tension force in a brace, allowing for load redistribution, P-delta and dynamic amplification effects
T_i	=	the pretension force in a wire brace
V	=	the total horizontal seismic force in the direction under consideration
V_T	=	the horizontal seismic force acting at roof level
W_x	=	that portion of W_t that is assigned to the level designated as x
W_t	=	the total reduced gravity load above the level of imposed lateral ground restraint
x, y	=	the principal axes of the cross-section
x_o, y_o	=	the coordinates of the shear centre of the cross-section
$\bar{x}$	=	the distance between the web and the y- axis
y_x	=	the height of the centre of mass of pallets above the pallet rail level designated as x
Z_x, Z_y	=	the section moduli about the x- and y- axes

α	=	the angle of inclination of a brace to the line of action of the applied force
β	=	a factor in the expression for deflection of a wire or cable braced column
Δ	=	deflection
η	=	the imperfection parameter
γ	=	a factor in the expression for K^1 to take account of available structural damping
θ	=	column base rotation or joint rotation
θ_c	=	the stability coefficient in terms of reaction forces
θ_e	=	floor irregularity
θ_m	=	the stability coefficient in terms of bending moments
ϕ	=	the capacity reduction factor
Ω	=	the load factor for compression members

13 APPENDICES

Appendix 1	:	"SM" or "S_pM_p" Factors for Steel Pallet Racks
Appendix 2	:	Brace Elasticity
Appendix 3	:	Column End Restraints
Appendix 4	:	Allowable Axial Load in Upright Frame Column
Appendix 5	:	Design Example for Drive-In Storage Rack
Appendix 6	:	Schedule of Essential Information To Be Completed By Rack Purchasers/Specifiers

APPENDIX 1

"SM" or "S_pM_p" FACTORS FOR STEEL PALLET RACKS
(Adapted from DG 8.3 Table 5P [3])

SM (or S_pM_p) factors have been assessed assuming M = 0.8 for steel, and allocating values to NZS 4203 structural type factor S according to ductility capability and assumed equivalent viscous damping levels, as discussed in reference [1].

SM, or S_pM_p

CATEGORY A: - Shear Walls and Diaphragms

These types apply to diaphragms or shear walls of steel, or other sheet material, which are subject to specific design under relevant materials codes.

Type *A1	Ductile	1.2
Type A2	Limited Ductile	2.4

CATEGORY B: - Moment Resisting Frames

Category "B" frames apply in situations where it is not practical to add supplementary bracing to the gravity frame system, due to restrictions of the material handling function on the structural form.

Type *B1 Frames, with Ductile Connections — 1.2

These frames assume numerous and widely distributed semi-rigid joints, with adequate strength reserve and high damping. Full Scale tests, as reported in reference [16] measured damping from shaking table motion decay data in the range 4 to 6% for frame deformations up to 0.04H, when subjected to approximately half E1 Centro 1940 N-S acceleration.

Fasteners for semi-rigid joints in this frame type should be designed to provide:

(i) Deformations consistent with the level of damping assumed to occur;

(ii) A margin of strength in the fastener design to permit a controlled mechanism of deformation to occur under seismic overload.

Type *B2 Frames, with semi-rigid connections of limited ductility — 2.4 to 3.2

Numerous and widely distributed joints with a damping range of 3 to 4% have been assumed in these frames, with deformation up to 0.02H.

Type B3 Non-Ductile Frames — 4.0

		SM, or S_pM_p

CATEGORY C: - Diagonally Braced Upright Frames

Type *C1(a)	Eccentrically Braced Frames (EBF)1.2 *Design and detailing of EBF's shall be in accordance with NZS3404: Part 2, Clause 12.11, or shall be experimentally verified.*	
Type *C1(b)	Concentrically Braced Frames (CBF) with braced members capable of plastic deformation in:	
	(i) Tension	2.0
	(ii) Tension and Compression	1.2 to 2.0
	Bracing would normally be fabricated from Grade 250 mild steel, with capacity designed connections and stable compression members. Design and detailing shall be in accordance with NZS3404: Part 2, Clause 12.12, or shall be experimentally verified.	
Type *C2	Braced members capable of acting in tension or compression with:	
	(a) Ductile End Connections	1.6
	(b) End Connections having Limited Ductility	2.0
	Evaluation of S factors for upright frames fabricated from cold formed channels depend on strength and stiffness degradation in the deforming eccentric brace to column connections under cyclic loading. This characteristic should be assessed experimentally in accordance with Part 4 of DG 8.3. *The SM value of 1.6 is designed to ensure minimum initial stiffness for these frames.*	
Type C3	Non-Ductile Brace Members	3.2
	The limiting design condition for non-ductile upright frames with aspect (H/D) ratios between 2 and 3 is normally one leg uplifting.	
Type C4	Upright Frames Detailed for Rocking Response	Special Study
	For rigid upright frames of aspect ratio H/D, uplift and rocking response will occur for ground accelerations exceeding 0.75/(H/D). Approximate methods for predicting period and amplitude of rocking response are given in references [1], [17]. Elastic distortions occurring within the frames should be added to displacements calculated for rocking. Baseplates, and anchorage connections on upright frames designed to respond in this manner should be detailed to permit the expected uplift to occur. All frame members should be proportioned to have a dependable strength adequate to support the unfactored design gravity loads with one leg uplifted, allowing for an impact factor of 1.15.	

		SM, or S_pM_p
CATEGORY D: - Supplementary Bracing Acting with Gravity Frames		
	Provision of bracing is recommended wherever possible to supplement the lateral force resistance of semi-rigid moment frames. SM factors derived for these systems are based on ductility, or the energy absorption capacity of the bracing system itself, or on damping available in semi-rigid joints under a specified frame deformation, or a combination of these.	
Type D1	Elastic Bracing	4.0
	There is no frame deformation compatibility requirement for bracing of this type.	
Type D2	Elastic bracing acting in tension with moment frames with semi-rigid connections of limited ductility.	
	(a) Limited Number of Connections	3.0
	(b) Numerous Connections	2.0
	This system presumes high strength steel bracing e.g. cable, or tendon, with sufficient flexibility to mobilise deformations up to 0.04H. Type D2(a) applies for drive-in racks where moment resisting connections are provided between columns and back tie and top tie members. Type D2(b) would normally apply for standard pallet racks, although drive-in racks with anchor frames may also be included in this category provided that the anchor frames carry not less than 25% of the total seismic force in the rear plane.	
Type D3	Brace members, capable of plastic deformation in tension	1.6
	Normal tension bracing S factors apply to this type of system. Due to low deformations no enhancement is available from the rack frame, regardless of connection type.	
Type *D4	Bracing members incorporating energy absorbing element of approved type.	1.0 or Special Study
	A suitable energy dissipating element for bracing of this type is the rectangular yielding "ring", as described in References [10] and [11].	

NOTES:

(1) The adequacy of frame systems, or member connections denoted (*) to be experimentally verified in accordance with Clause 4.1 of DG 8.3 [3].

APPENDIX 2

BRACE "ELASTICITY"

The choice of bracing type and level of prestress in wire/cable systems can have a significant effect on the deflection of a structure under lateral load. This is illustrated in the following worked example, comparing mild steel bracing rod with diagonally opposed 7mm BBR prestressing wire at differing levels of pretension.

A2.1 Deflection Calculations:

Problem Statement:

Calculate the horizontal deflection at the top of the braced column shown in Fig. A2.1 (a), for bracing type:

(a) Mild Steel Bracing Rod;
(b) Pretensioned 7 mm BBR Prestressing Wire

Given:

P_u = 30 kN (load level for "strength method" of design)

a = 9.128 m

b = 6.9 m

Calculate:

α = 37.09°

L = 11.44 m

(a) Mild Steel Bracing

Brace load, T = 30/cos (37.09)
= 37.6 kN

Required Brace Area:

A_b = T/F_y
= 100 mm² for F_y = 250 MPa
and EA_b = 20,000 kN for E = 200,000 MPa

Top Deflection = $P_uL/EA \cos^2\alpha$
= 30.0 x 11.44/(20,000 x 0.64)
= 0.027 m (27 mm)
based on minimum brace area for the given loading.

(b) 7 mm BBR Prestressing Wire Bracing

Wire deflects under its own self weight (refer Fig A2.1 (b)), and column top deflection for a given loading depends on wire self weight as well as the level of pretension in the bracing wires. A solution is readily obtained using manual methods [18], or by use of a suitable non-linear computer analysis programme. In the case of light bracing wire of relatively short length (which will apply for most drive-in rack structures) the wire sag deflection due to self weight may be ignored, which greatly simplifies the deflection calculation. The effective bracing stiffness may then be represented by a simple bilinear relationship, as follows:-

	K	=	$2EA\cos^2\alpha/L$ for $P \leq 2T_i\cos\alpha$
and	K	=	$EA\cos^2\alpha/L$ for $P > 2T_i\cos\alpha$

The corresponding column top deflection for any level of applied load or wire prestress is given by:-

$$\Delta = \beta L/(2EA\cos^2\alpha)$$

where	β	=	P for $P \leq 2T_i\cos\alpha$
and	β	=	$2P - 2T_i\cos\alpha$ for $P > 2T_i\cos\alpha$

For the example in hand, we have:-

		A	=	38.5 mm^2
		EA	=	7,700 kN for E = 200,000 MPa
		α	=	37.09°
		L	=	11.44 m
P	=	P_u	=	30 kN
say		T_i	=	10 kN
then	$2T_i\cos\alpha$		=	15.95 kN
		β	=	60 - 15.95 = 44.05
and		Δ	=	44.05 x 11.44/(2 x 7, 700 x 0.64)
			=	0.051 m (51 mm)

A plot of column top deflection versus wire pretension, as determined from a non-linear computer analysis for an applied load of 30 kN, is presented in Fig A2.2. This gives Δ = 52.2 mm for a wire pretension of 10 kN, which is in good agreement with the result obtained above using the simplification of weightless wire. The deflection value of about 52mm compares with 27 mm for a mild steel brace of approximately equivalent strength.

A2.2 Wire Factor of Safety

Check wire factor of safety, given a wire pretension of about 10 kN.

At strength level, T = 37.8 kN (37.6 kN for weightless wire)

Therefore, at "working load" level,

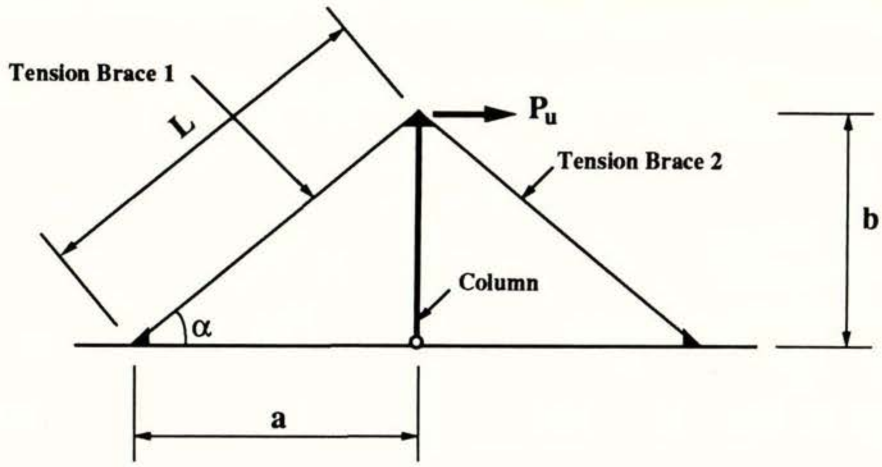
$$T = 0.8 \times 37.8 = 30.2 \text{ kN.}$$

The breaking strength of 7 mm BBR strand is 64 kN.

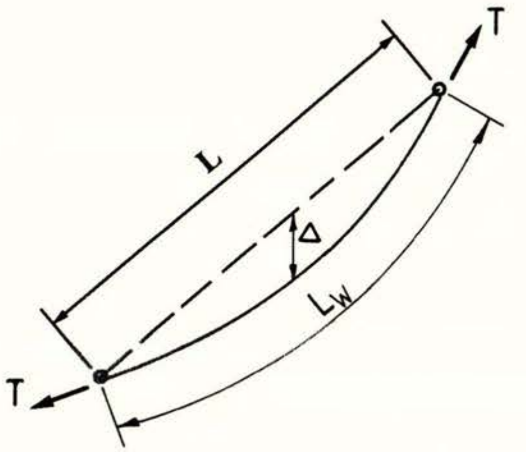
Therefore FOS = 64/30.2
= 2.1 OK

A2.3 Graphical Comparison of Mild Steel and Wire Bracing

Load-deflection plots for the bracing elements calculated in the preceding section are presented in Fig. A2.3. This figure shows graphically the effect of wire pretension and bracing type on load deflection behaviour. For wire bracing systems, bracing stiffness will normally fall within the range $EA\cos^2\alpha/L$ to $2EA\cos^2\alpha/L$. The actual "effective stiffness" depends on the applied load as well as the amount of wire pretension.



(a) Column and Brace Layout



(b) Wire Deflection under Self-Weight

Fig. A2.1 Braced Column For Deflection Calculation

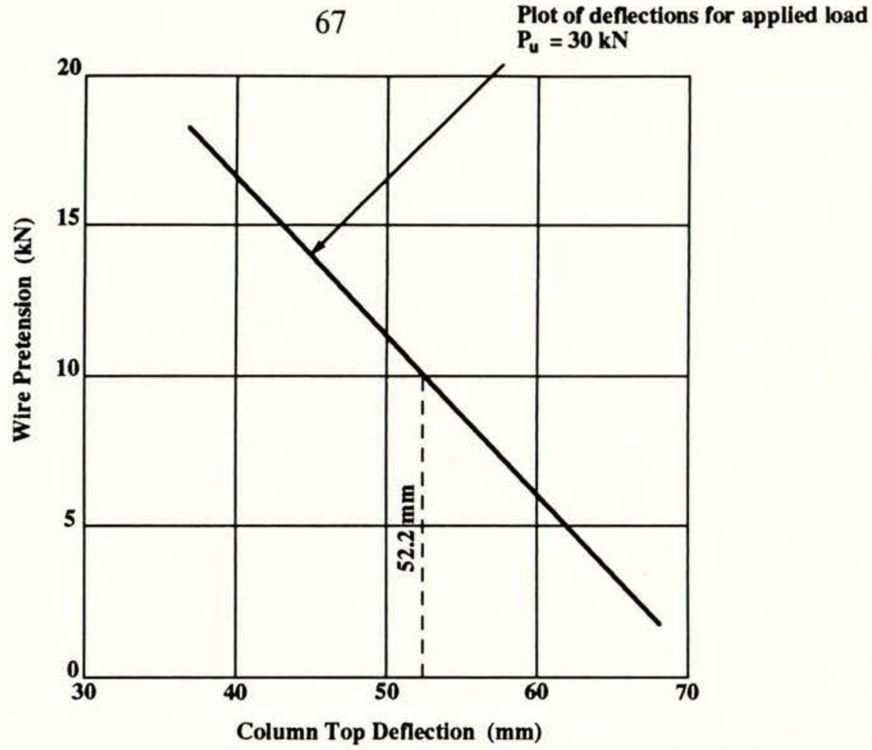


Fig. A2.2 Column Top Deflection vs Wire Pretension

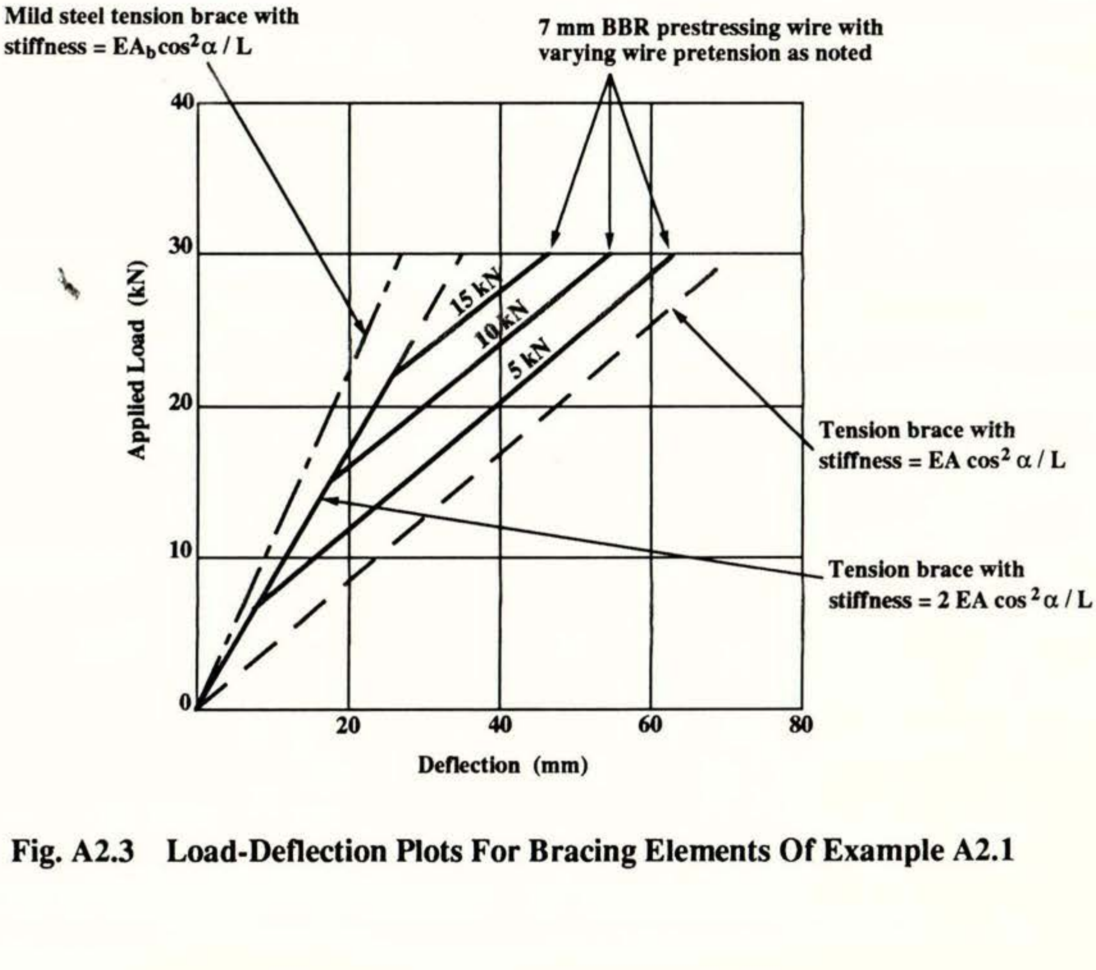


Fig. A2.3 Load-Deflection Plots For Bracing Elements Of Example A2.1

APPENDIX 3

UPRIGHT COLUMN END RESTRAINTS

A3.1 Top Rotational Restraint

Consider a typical drive-in rack loaded in the longitudinal direction (Fig. A3.1(a) refers). Since the top tie beam will go into double bending under the action of lateral loading, a single interior column may be represented as shown in Fig. A3.1(b).

Using the moment-rotation relationship depicted in Fig. A3.1(c), the top tie beam may be replaced by a rotational spring of stiffness:-

$$K_b = M/\theta = 2[3EI_b/(L_b/2)]$$
$$= 12EI_b/L_b$$

To allow for a semi-rigid beam to column connection, beam stiffness may be reduced accordingly to the relationship:-

$$\left(\frac{I_b}{L_b}\right)_{reduced} = \left(\frac{1}{1+6EI_b/K_\theta L_b}\right)\left(\frac{I_b}{L_b}\right)$$

K_θ is the joint rotation stiffness which must be determined from test. Values for K_θ from tests by Chen et al [16] on proprietary rack beam/column joints typically fell within the range 500 - 1000 kip-in/radian (56 - 112 kN-m/radian).

A3.2 Column Base Rotational Restraint

A loaded column bearing on a flat concrete floor develops limited base rotational stiffness, which will vary depending on column dimensions, base-plate stiffness, holding down bolt arrangement and ratio of applied moment to column axial load.

For unbolted bases on flat concrete floors Stark and Tilburgs [19] proposed that K_f = 50 kN-m/radian be adopted as a good lower limit estimate of base stiffness, based on component tests on unbolted footings.

For a bolted connection with a thin base-plate the expression for base stiffness is given as:-

$$K_f = db^2E_c/12$$

where the outside column dimension is d x b [19].

Chen et al [16] used fictitious floor beams connected at column bases to model base rotational restraint. They found that setting I_f = 0.2 in^4 (83,000mm^4) in their theoretical model gave good agreement with measured results. Depending on the "floor beam" length, and number of "floor beams" in the model, the equivalent average base rotational stiffness is 50-100 kN-m/radian, giving good agreement with the recommendation of Stark and Tilburgs.

A3.3 Column Top Translational Restraint

With reference to the bracing layout shown in Fig 4.3.1, the top translational spring consists of rear plane and roof level diaphragm bracing components.

(a) Rear Plane Bracing:

As discussed in Section A2.3, brace stiffness may be taken to fall within the range $EA\cos^2\alpha/L$ to $2EA\cos^2\alpha/L$ for a pair of tensioned cross-braces. The minimum and maximum values for the rear plane bracing component per column may be readily calculated from these expressions, assuming that the effect of frame action in the rear plane is small (normally the case, unless anchor frames are provided).

(b) Roof Level Diaphragm Bracing:

The roof level diaphragm "bracing" component will vary depending on column location, bracing configuration and details of rack longitudinal and transverse members. For wire bracing systems it is recommended that a model such as TDI (refer Section 6.4) be used to predict the roof level diaphragm "bracing" component. If approximate methods are used, these should err on the conservative side.

(c) Bracing Stiffness for Column Stability:

The net "effective" bracing stiffness should normally be adequate to ensure that the column is, in effect, "sway prevented". This may be checked by considering the equilibrium of the deflected column under the action of P-delta forces and the restraining spring force. By making some simplifying assumptions as shown in Fig. A3.2, an estimate of the maximum spring stiffness required is readily obtained, as:-

$$K = 2P/H$$

where P = the total vertical loading on the column

and H = the height of the column

A3.4 Example Calculation of Upright Column End Restraints

A3.4.1 Given:-

- Top tie beam continuous and bolted to columns, with joint rigidity K_θ = 100 kN-m/radian (as determined experimentally).
- Tie beam I_b/L_b = 725mm^3
- Upright frames set on flat concrete floor, with one holding down bolt at each column position.
- P = 25.2 kN
- H = 6.9 m

A3.4.2 Top Rotational Restraint:

$$\left(\frac{I_b}{L_b}\right)_{reduced} = \left(\frac{1}{1+6EI_b/K_\theta L_b}\right)\left(\frac{I_b}{L_b}\right)$$

$$= (0.103) \times 725$$
$$= 75 \text{ mm}^3$$

then K_b = $12E\,(I_b/L_b)$ reduced
= 180 kN-m/radian

A3.4.3 Base Rotational Restraint:

Adopt K_r = 50 kN-m/radian

A3.4.4 Top Translational Spring:

A. Rear Plane Bracing Component:

The rear plane bracing consists of 8 pairs of wires, or one pair/17.5 interior columns.

Wire	EA	=	7,700 kN
	L	=	11.44 m
	α	=	37.09°

With reference to Section A2.3 and Fig. A2.3;

Min. Brace Stiffness	=	$EA\cos^2\alpha/L$
	=	430 kN/m (25 kN/m per column)
Max. Brace Stiffness	=	860 kN/m (50 kN/m per column)

For a roof diaphragm lateral load of 190 kN (from Section A5.5(C) in Appendix 5), the lateral load/pair of wires is about 24 kN. For a pretension force of 10 kN, the corresponding brace stiffness is approximately 600 kN/m (35 kN/m per column).

B. Roof Level Diaphragm Bracing Component:

Refer Fig. 6.4.2 for output data from the TDI computer model, for the case of I_y = 0.1 x 10^6mm^4. The minimum and maximum values of the roof level deflection component for an interior column are 17 mm and 98 mm respectively. The corresponding spring stiffnesses are approximately 75 kN/m and 12.5 kN/m.

C. Effective Brace Stiffness/Column:

Combining spring stiffnesses from A and B;

Min. Stiffness	=	(25 x 12.5)/(25 + 12.5) = 8.3 kN/m
Max. Stiffness	=	(50 x 75)/50 + 75) = 30 kN/m

Notes:

(1) From Section A3.3(c) the minimum brace stiffness for column stability is estimated to be:-

$$2P/H = 50.4/6.9 = 7.3 \text{ kN/m}$$

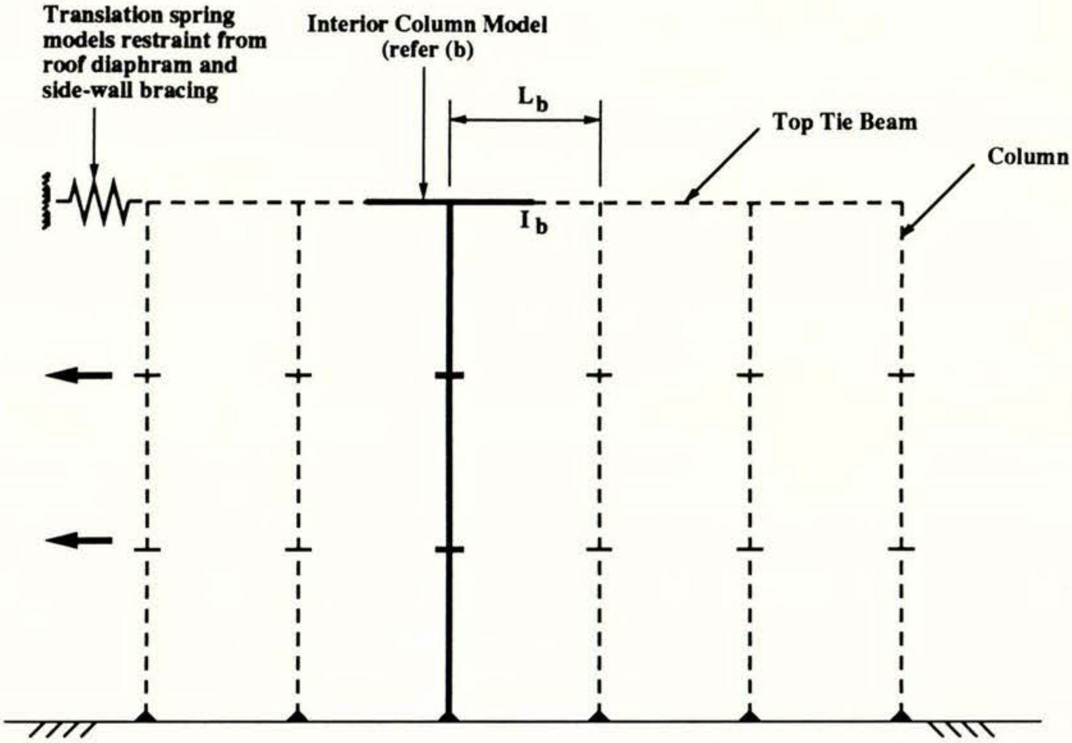
This is less than the minimum stiffness of 8.3 kN/m as calculated above, and the columns may be taken to be "sway prevented".

(2) If the rear plane bracing stiffness component is taken to be 35 kN/m per column (refer (A)), then the minimum effective brace stiffness per column becomes:-

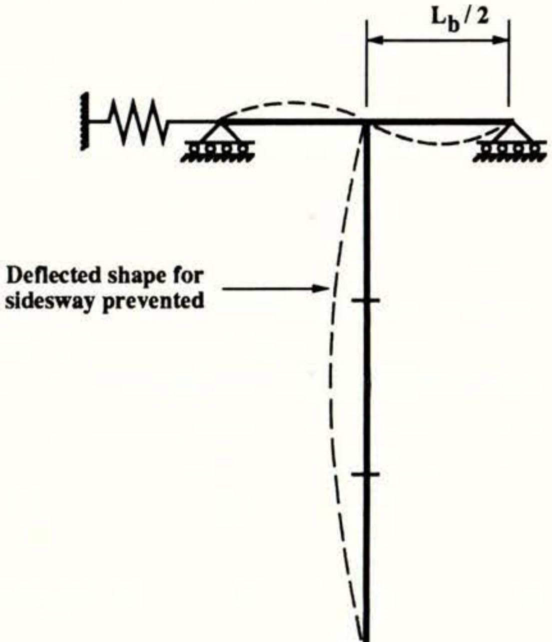
$$(35 \times 12.5)/(35 + 12.5) = 9.2 \text{ kN/m}$$

Further, a global load reduction factor of 0.75 applies to brace loading (as a brace supports many columns), but not to the loads on a single column. Thus, the "effective" minimum stiffness of the translational spring support to a single column becomes:-

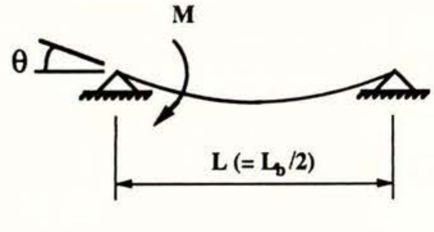
$$9.2/0.75 = 12.3 \text{ kN/m}$$



(a) Front Elevation of Drive-In Rack Loaded in the Longitudinal Direction
(pallets not shown)



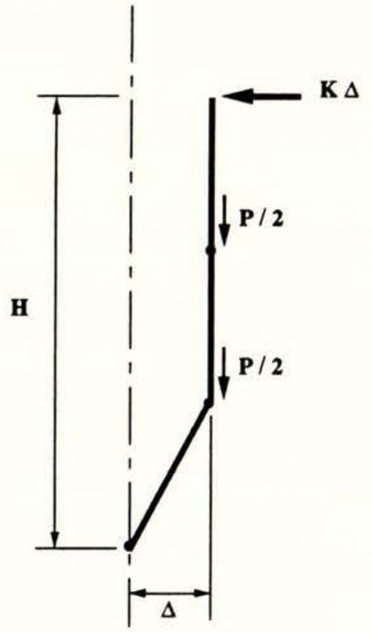
(b) Interior Column Model



$M / \theta = 3\,EI / L$

(c) M - θ Relationship

Fig. A3.1 Column End Restraints



For a deflected shape as shown, and pinned ends, the equilibrium equation is :-

$$P \Delta = K \Delta H$$

ie $K = P / H$

Applying a factor of safety of 2, a minimum spring stiffness of

$$K = 2P / H$$

will normally ensure adequate support to the top of a column to allow it to be considered as "sway prevented"

Fig A3.2 Stability of Column under Longitudinal Load

APPENDIX 4

ALLOWABLE AXIAL LOAD IN UPRIGHT FRAME COLUMN

A4.1 General

- Calculations generally to AS1538 [20]
- Column length 6.9m
- Upright frame bracing fixed to column at 1.38m centres
- Column end restraints as example in Clause A3.4
- Column Section properties as given on Fig. A4.1

Note:

It is assumed that the column is not perforated. Where a column is perforated, reference should be made to Section 3 of the RMI Specification [21] for modifications to formulae to take account of perforations. These relate particularly to the form factor Q (which must be determined experimentally) and the use of the minimum net cross-sectional area of the column in determining allowable loads.

A4.2 Column Effective Lengths (the local x-x axis is in the plane of the upright frame).

A. y- Axis Flexural Buckling:

The column is supported at 1380 crs

$\therefore \quad l_y/r_y = 1380/23.1 = 59.7$

(Note - for bracing requirements refer Clause 5.2.2 of AS1538).

B. x- Axis Flexural Buckling:

The column is supported top and bottom, L = 6.9 m

(i) Reduction factor for end restraints:-

(Refer Appendix E of AS1250 [5] and Section A3.4 of Appendix 3).

$I_c/L_c = 2.08 \times 10^6/6.9 \times 10^3 = 301 \text{ mm}^3$
$\Sigma(I_b/L_b)_{red.} = 2 \times 75 = 150 \text{ mm}^3$ (from Section A3.4)

$$G_B = 301/150 = 2.0$$

Also, $G_A = 2.0 \times 180/50 = 7.2$ for base spring stiffness of 50 kN-m/radian

Fig. E1 in AS1250 gives $l/L = 0.90$.

(ii) Reduction factor for column loaded along its length:

Refering to Table 34 in Roark [22] it can be shown from Case 3a that the reduction factors for a pinned ended column are:

0.82 for 33% load at top and remainder uniformly distributed
0.73 for all load uniformly distributed over full length of column
- say adopt 0.80 for a typical rack column loading.

Combining (i) and (ii), $l_x/L = 0.9 \times 0.8 = 0.72$

Therefore $l_x = 0.72 \times 6900 = 4970$

and $l_x/r_x = 4970/49.4 = 100.6$

C. **Torsional Buckling:**

$L = 1380$ (assumes twisting of column prevented at brace points)

$K_T = 0.8$ (Clause 5.3.3 in [21])

Therefore $l_z = 0.8 \times 1380 = 1100$ mm

A4.3 Determination of Elastic Buckling Stress, F_{oc} (Clause 3.6.4 of AS1538)

$F_{oy} = \pi^2 E/(l_y/r_y)^2 = 554$ MPa

$F_{ox} = \pi^2 E/(l_x/r_x)^2 = 195$ MPa

$F_{oz} = GJ\,(1 + \pi^2 EI_w/GJl_z^2)/A(r_{01})^2$

$= 597$ MPa when $G = 80{,}000$ MPa and $l_z = 1100$mm.

$$F_{oxz} = \frac{(F_{ox}+F_{oz})-\sqrt{[(F_{ox}-F_{oz})^2+4F_{ox}F_{oz}(x_o/r_{01})^2]}}{2[1-(x_o/r_{01})^2]}$$

$= 165$ MPa

F_{oc} = smaller of F_{oxz} and F_{oy}

$= 165$ MPa

A4.4 Calculate Max. Permissible Load, P_a (Clause 3.6.1 of AS1538)

$F_{oc}/QF_y = 165/280 = 0.59$ for $F_y = 280$ MPa

$\eta = (1.25 - Q)\,QF_y/F_{oc}$

$= (1.25 - 1.0) \times 1.0 \times 280/165 = 0.424$

Substitution in equation 3.6.1 gives:-

$F_a = (QF_y/\Omega)(0.413) = 69.4$ MPa where $\Omega = 1/0.6$

$P_a = 69.4 \times 854 = 59.3$ kN

Dependable Compressive Strength:-

$= \phi\, P_a/0.6$

$= 88.9$ kN for $\phi = 0.9$ - (Refer Clause C.2.1 of DG 8.3 [3])

<u>Column Section Properties</u>

I_x = 2,080,000 mm^4

I_y = 455,580 mm^4

Z_x = 33,280 mm^3

Z_y = 11,110 mm^3

r_x = 49.4 mm

r_y = 23.1 mm

A = 854 mm^2

Q = 1.0

$\bar{x}$ = 20.4 mm

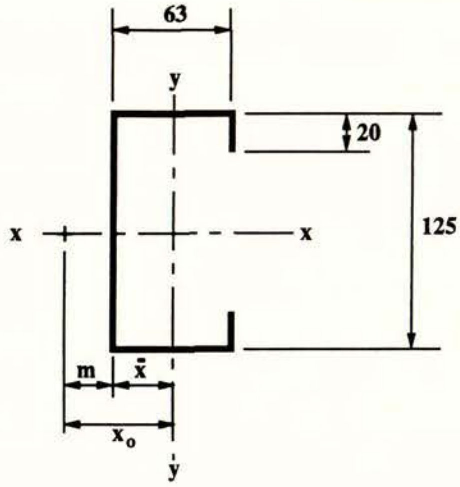
m = 32.6 mm

J = 2915 mm^4

$I_w = 1.66 \times 10^9 mm^6$

x_o = 53 mm

r_{o1} = 76.0 mm



t = 3.2 mm
R = 5.0 mm

Fig. A4.1 Column Section Properties

APPENDIX 5

DESIGN EXAMPLE FOR DRIVE-IN STORAGE RACK

A5.1 **Geometry**

Rack layout is as shown in Figs. 4.3.1 and 4.3.2.

A5.2 **Assumptions**

(i) Location Zone 'B' on rigid subsoils.

(ii) Standard gravity frame in cold formed steel.

(iii) Seismic resistance to be provided by:

- Transverse Direction:
  Braced upright frames anchored to floor.

- Longitudinal Direction:
  Elastic bracing acting in tension with moment frames with limited number of semi-rigid connections.

A5.3 **Loadings**

Storage capacity 9 No. 1000 kg pallets/level/bay.

Pallet rails at heights 2.3 m and 4.6 m
Centres of mass at heights 3.3 m and 5.6 m

A5.4 **Earthquake in Transverse Direction**

A. **Seismic Design Coefficient: Transverse**

Take SM = 2.0 - (Type C2(b) in Appendix 1 - It is assumed that the upright frame has been tested to verify post elastic performance).

From Table 6.2.1 the frame first mode period is 0.42 seconds.

Therefore C = 0.125 (Zone B, rigid subsoils, short period structure).

Now C_d = 0.7 CSM (Clause 3.2.1 in DG 8.3[3])
= 0.7 x 0.125 x 2.0
= 0.175

B. **Seismic Loading & Distribution**

The reduced gravity load/level/bay:-

= 0.75 x 9 No x 1000 kg (Clause C.3.3 in DG 8.3 [3])

= 66 kN

(Total reduced gravity load = 66 kN x 2 x 20 = 2640 kN)

Base shear/bay = 0.175 x 66 x 2 No levels
= 23.1 kN

Distribution of Horizontal Forces:-

Level	h_x (m)	W_x (kN)	$W_x h_x$	$\frac{W_x h_x}{\sum W_x h_x}$	F_x (kN)	$F_x h_x$
4	6.9	-	-	-	-	-
3	4.6	66	303.6	0.67	15.4	70.8
2	2.3	66	151.8	0.33	7.7	17.7
		Totals	455.4	1.00	23.1	88.5

C. Column Loads

Moment/frame = 88.5/4 = 22.1 kN-m

and corresponding leg load = 22.1/1.2 = 18.4 kN (interior columns)
x 1.35 overstrength factor = 24.9 kN.

The additional load in exterior columns due to pallet centres of mass above rail levels is:-

P_E = 23.1/8.16 = 2.8 kN for $y_2 = y_3 = 1.0$ m.

Therefore Max. E/Q induced leg load = 21.2 kN (exterior columns)
x 1.35 overstrength factor = 28.6 kN

Axial load due to stored loads = 2 x 9000 kg/7 No. columns
= 25.2 kN (interior columns)
or 12.6 kN (exterior columns)

Max. Column Uplift (refer Clause C3.4.1 in DG 8.3 [3]):
0.9D/3 + E = 0.9 x 12.6/3 - 28.6 = -24.8 kN (exterior column).

Max. Column Load:
D + E = 25.2 + 24.9 = 50.1 kN (interior column)

A5.5 Earthquake in Longitudinal Direction

A Seismic Design Coefficient: Longitudinal

Take SM = 3.0 (Type D2(a) in Appendix 1).

C = 0.0625 (Zone B, rigid subsoils, structure period > 1.2 secs)

Note: Actual structure period, T_1 = 1.58 secs (from Section 6.5.3 for the RACK10 model with column end rotational restraints). This applies for a reduced gravity load of 2/3 L on the overall rack structure, but with full gravity loading locally on a column. For lesser levels of rack loading the period will reduce, and response should be checked for a global load level of, say, 1/3 L. When designing to NZS4203 [4] this check is only required if structure period is likely to drop below 1.2 seconds.

C_d = 0.7 CSM
= 0.131

B Seismic Loading & Distribution

Design gravity load/level/column - 9000 kg/7 No. = 12.6 kN.

Therefore total seismic load/column = 0.131 x 12.6 x 2
= 3.3 kN

Distribution of Horizontal Forces:-

Level	h_x (m)	W_x (kN)	W_xh_x	$\frac{W_xh_x}{\sum W_xh_x}$	F_x (kN)
4	6.9	-	-	-	-
3	4.6	12.6	58.0	0.67	2.2
2	2.3	12.6	29.0	0.33	1.1
	Totals		87.0		3.3

C Loading in Roof Level Diaphragm

From Section A5.4(B), the total reduced gravity load,

W_t = 2640 kN
and V = 0.131 x 2640 = 346 kN

Assume that 55% of V is transferred into the roof level diaphragm (corresponding approximately to the case where the column is pinned both ends, and is a likely maximum value).

Then the longitudinal load at roof level (Level 4) is:-

V_T = 0.55 x 346 = 190 kN

Refer Section 6.4 for the roof level diaphragm computer model and Fig. 6.4.2 for analysis results.

D Column Bending Moments

Column bending moments may be calculated from a simple column model such as COL10 (Section 6.3), or from a grillage model such as RACK10 (Section 6.5). Column end restraints may be calculated as detailed in Clause A3.4 of Appendix 3. Note that a higher C (and hence C_d) factor may apply for local column loading effects when the overall rack structure is only lightly loaded. Refer note in Section A5.5(A).

E Column Axial Loads

(a) Gravity Loads:

From (B) above:-

D + L = 12.6 kN at each of levels 2 and 3
= 25.2 kN total on a single column

(b) Earthquake Induced Loads

To the gravity loads must be added an earthquake load arising from the action of the upright frames in providing torsional restraint to the rack as a whole under longitudinal loading.

From Fig. 6.4.2, the maximum load in a line of upright frames is 24.8 kN. Allowing for possible eccentricity of loading and P-delta effects, the design lateral load applied to the top of a line of upright frames is:-

$$P_u = K_e K_b \times 24.8$$

Now, a loading eccentricity of + 0.1B represents a 20% increase in the applied torsional moment.

Therefore, $K_e = 1.2$.

K_b may be obtained from a P-delta analysis as described in Section 8.3, given that the average column top deflection (from Fig. 6.4.2) is about 90mm, and average column loading is 9.5kN at levels 3 and 2.

We have:-

Level	Average Column Loads (kN)	Approx. Ave. Deflections (mm)	Average R (kN)	K_c
4	-	90	1.37	1.18
3	9.5	95	-	
2	9.5	60	-	

where
$$R_i = 9.5 \times (.095 + .06)/6.9 = 0.21$$
$$\theta_c = R_i/R = 0.156$$
$$K_c = 1/(1-\theta_c) = 1.18$$

Thus, $K_b = K_c = 1.18$, say 1.2

Therefore,
$$P_u = 1.2 \times 1.2 \times 24.8$$
$$= 35.7\text{, say } 36 \text{ kN}$$

By inspection, this will give an excessive loading in the upright frames (compare base shear/bay = 23.1 kN from Section A5.4 (B)), and supplementary cross-bracing is required, as shown on Fig. 4.3.2. Note that, with the bracing arranged as shown, the column member which is loaded in compression by the tension brace is simultaneously loaded in tension as a result of the upright frame action in carrying a portion of the applied lateral loading. The optimum design is when the vertical component of brace load is twice the vertical load in an upright frame column leg resulting from brace action alone. Thus, there is an optimum ratio of brace to frame stiffness, which may be determined as follows:-

We have: Brace Length $L = 9.0$ m
$$\alpha = 50°$$

Let component of P_u carried by frame action = P_f

Then the corresponding load in a column leg $= P_f \times 6.9/(4 \times 1.2)$
$$= 1.44 P_f$$
(since there are 4 frames of height 6.9 m and width 1.2 m).

Also, the vertical component of brace load = $(36-P_f) \tan\alpha$.

Now, design is optimum when $(36-P_f)\tan\alpha = 2 \times (1.44 P_f)$
giving $P_f = 10.5$ kN or about 29% P_u.

Therefore bracing is to carry 71% P_u, and bracing stiffness needs to be about 2.5 times frame stiffness, that is, 2.5 x 1000 kN/m = 2,500 kN/m (refer Section 6.4.1).

(i) Assume rod bracing of stiffness $EA\cos^2\alpha/L$.

Calculate, A = 2,500 x 9.0/(0.413 x 200)

= 264 mm² for K = 2,500 kN/m

Say 2 pairs of M12 bracing rods with A = 226 mm² and actual bracing stiffness of 2,140 kN/m giving a revised P_t = 11.5 kN

Column load from frame action = 1.44 P_t = 16.5 kN

Vertical component of bracing load = $(36-P_t)\tan\alpha$ = 29.2 kN

∴ Maximum induced vertical load in a column leg = 16.5 kN.

Also, bracing load = $(36-P_t)\cos\alpha$ = 38.1 kN (at strength level)

= 30.5 kN at "working" level

The permissible load in 2/M12 screwed tension rods under earthquake loading = 2 x 1.33 x 12.1 kN = 32.2 kN > 30.5 kN - O.K.

(ii) Assume pretensioned 7 mm BBR wire bracing of stiffness $2EA\cos^2\alpha/L$

Calculate A = 132 mm² for K = 2,500 kN/m

Therefore 4 pairs of wires required, with A = 154 mm² and actual bracing stiffness of 2,920 kN/m giving, in this case, P_t = 9.2 kN.

Column load from frame action = 1.44 P_t = 13.2 kN

Vertical component of bracing load = $(36-P_t)\tan\alpha$ = 31.9 kN

and the maximum induced column load = 31.9 - 13.2

= 18.7 kN

The bracing load = $(36-P_t)/\cos\alpha$ = 41.7 kN
or 10.4 kN/pair of bracing wires - O.K.

In this case assume that rod bracing is used and that the earthquake induced column load is 16.5 kN. Note that most upright frame lines will require at least one pair of bracing rods if the earthquake induced column load is to be limited to be less than 16.5 kN in all cases. The design of the transverse bracing and effect on upright frame loads and base reactions should be checked for the case of earthquake in the transverse direction.

F P-delta Amplification of Column Moments and Reactions

Calculation of the amplification factors may be summarised as follows:-

Level	Col. Loads (kN)	Deflec -tions (mm)	M_{ix} (kN-m)	M_{ex} (kN-m)	K_{mx}	M_{nx} (kN-m)	R (kN)	K_c
4	16.5	124					1.61	1.64
3	12.6	116	1.36	3.74	1.80	6.73	-	-
2	<u>12.6</u>	70	1.47	<u>2.38</u>	1.92	4.58	-	-
Total	41.7		M_m =	3.06				

where:

- the column loads are obtained from (E)
- the deflection, R and M_{ex} values are obtained from Fig. 6.5.3 for the case of column with end rotation restraints
- the induced column top reaction R_i, and bending moments M_{ix}, are calculated by taking moments about the column base, as follows:-

(i) $R_i.L \; = \; \Sigma(P_x\Delta_x)$ where P_x = the vertical force applied at level x

$\therefore R_i \; = \; (16.5 \times .124 + 12.6 \times .116 + 12.6 \times .070)/6.9 = 0.63$ kN - approximate, as end rotation restraints would modify this.

(ii) $M_{i2} \; = \; P.\Delta_2 - R_i h_2$ where $P = \Sigma P_x$

$= (41.7 \times 0.07) - (0.63 \times 2.3) = 1.47$ kN-m

(iii) $M_{i3} \; = \; P.\Delta_3 - P_2(\Delta_3 - \Delta_2) - R_i h_3$

$= (41.7 \times .116) - (12.6 \times .046) - (0.63 \times 4.6) = 1.36$ kN-m

- the amplification factors are calculated as follows:-

$K_{mx} \; = \; 1/(1-\theta_m)$ where $\theta_m = M_{ix}/M_m$

$K_c \; = \; 1/(1-\theta_c)$ where $\theta_c = R_i/R$

<u>Note:</u>

The effect of column end restraint is illustrated by comparison of the above results with M_{nx} values computed from the data in Fig. 6.3.2 for a column with top translation spring stiffness of 10 kN/m.

This gives:-

	Pinned Ended Column	Column with Rotation Restraint Each End
M_{n3}	7.90	7.36
M_{n2}	7.32	4.80

The non-linear moments for the 3 cases show variations of 17% and 60% at levels 3 and 2 respectively. It is apparent that the base rotational restraint in particular has a significant effect on the design bending moments. Check that the column base moment does not exceed M_{max} given by:

$$\begin{aligned} M_{max} &= P.e \text{ where } e = b/2 \text{ (refer Fig 4.2.1)} \\ &= 41.7 \text{ kN} \times 0.0625\text{m} \\ &= 2.6 \text{ kN-m} \end{aligned}$$

This is significantly greater than M_{e1} = 1.5 kN-m (from Fig 6.5.3) indicating that the required base restraint can be developed.

G Dynamic Amplification of Column Moment:

Applying a dynamic amplification factor of 1.5 (refer Section 7.0), the design moments at levels 3 and 2 become:

$$\begin{aligned} M_{a3} &= 1.5 \times 6.73 = 10.10 \text{ kN-m} \\ M_{a2} &= 1.5 \times 4.58 = 6.87 \text{ kN-m} \end{aligned}$$

For Z_x = 33,280 mm^3, the corresponding bending stresses, f_{bx}, are 303 MPa and 206 MPa

A5.6 Column in Combined Bending and Compression

(Refer Section 3.7 in AS1538[20])

A Design Vertical Load on Column:

At Level 3 = 29.1 kN (f_a = 34.1 MPa)
At Level 2 = 41.7 kN (f_a = 48.8 MPa)
From Appendix 4, F_a = 69.4 MPa

B Calculate Maximum Permissible Bending Stress, F_{bx}

The section is bent about a plane of symmetry (in major axis bending), hence Clause 3.7.3.2 applies, giving:-

$$F_{ob} = A r_{01} \sqrt{F_{oy} F_{oz}} / Z_x$$

$$= 854 \times 76.0 \quad \sqrt{554 \times 597} / 33{,}280$$

$$= 1{,}121 \text{ MPa}$$

(A, r_{01} and Z_x from Fig A4.1
F_{oy}, F_{oz} from Appendix 4)

From Clause 3.3.2:

$$F_b = (0.95 - 0.50\sqrt{F_y / F_{ob}})F_y$$

$$= 196 \text{ MPa for } F_y = 280 \text{ MPa} \qquad \text{-(1)}$$

From Clause 3.1

$F_b = 0.6F_y = 168$ MPa -(2)

$F_{bx} \quad = \quad$ lesser of (1) and (2) = 168 MPa

C Interaction Equation:

The applicable interaction equation is equation 3.7.1(1) in AS1538, modified to allow for loading at "strength" level. That is:-

$$f_a/F_a + f_{bx}/F_{bx} \leq \Omega\, \phi$$
$$\leq 1.5 \text{ for } \phi = 0.90 \text{ and } \Omega = 1/0.6$$

Note: that equation 3.7.1(2) does not apply, as second order effects have already been included in the bending stress calculations.

We have:-

Level	f_a/F_a	f_{bx}/F_{bx}	$f_a/F_a + f_{bx}/F_{bx}$
3	34.1/69.4 = 0.49	303/168 = 1.80	2.29
2	48.8/69.4 = 0.70	206/168 = 1.23	1.93

Therefore the column is 52% overstressed at Level 3. Options are:-

(i) Use a stronger column section.

(ii) Increase roof level diaphragm stiffness to reduce column top deflection (and hence P-delta effects) and dynamic effects.

A5.7 Wire Bracing

A Permissible Load:

The breaking strength of 7mm BBR strand = 64 kN

From Section 4.3.4, a minimum factor of safety of 2.0 applies for earthquake loading, giving a maximum permissible "working load" of 32 kN.

At "strength" level, permissible load = 32/0.8
= 40 kN

B Roof Level Diaphragm Bracing:

From Fig. 6.4.2, the maximum brace load, T, is 31.9 kN, based on a linear static analysis for a uniformly distributed loading.

The design load $T_a = K_b K_d K_e T$

where K_b = the brace load amplification for P-delta effects
= the average value of K_c for all columns supported.
= 1.3 (estimated - or may be calculated in a similar manner to the calculation of K_b for the overall structure in Section A5.5(E)).

K_d = the load amplification factor for dynamic effects
= 1.5 (from Section 7.0)

K_e = the load amplification factor to account for possible eccentricity of load distribution
= 1.2, say

Therefore, the overall amplification factor on diaphragm bracing load
= 1.3 x 1.5 x 1.2 = 2.34

And T_a = 2.34 x 31.9
= 74.6 kN

This is some 86% in excess of the permissible load of 40 kN. A redesign of the bracing is required to reduce the maximum brace load. Options include:-

(i) Provide additional pairs of bracing;
(ii) Modify structural details to achieve a more even distribution of loading to the bracing elements. This may also give a reduction of dynamic amplification effects in the critically loaded elements. A dynamic analysis would be required to confirm use of a dynamic amplification factor of less than 1.5.

C **Rear Plane Bracing:**

From Section A5.5(C), the design longitudinal load,

V_T = 190 kN

In this case, $K_d = K_e = 1.0$

The P-delta amplification factor is an "average" value as computed in Section A5.5 (E)

say K_b = 1.2

Bracing inclination is 37.1°

Therefore total brace load = (1.2 x 190)/cos37.1
= 286 kN

There are 8 pairs of braces, therefore the maximum load in a brace (assuming all braces are equally tensioned)

= 286/8
= 35.7 kN

This is less than the permissible load of 40 kN, so rear plane bracing is okay as detailed.

D **Wire Bracing End Connections:**

Design of end connections must comply with the relevant requirements of NZS3404[5] and AS1538[20]. If the method of securing the wire reduces the wire breaking strength, this must be allowed for in calculation of the permissible wire load.

A5.8 Upright Frame Members

A Brace Member in Upright Frame:

A	=	$260mm^2$
r_y	=	7.5mm
l	=	1,380

Base shear/bay = 23.1 kN (from Section A5.4(B))

$\therefore$ Base shear/frame = 23.1/4 = 5.8 kN

and brace load = 5.8/cos45	=	8.2 kN
x 1.35 overstrength factor	=	11.1 kN

Now l/r_y = 184

Therefore	F_a	=	28 MPa
and	P_a	=	7.3 kN

Dependable strength = ϕ7.3/0.6 = 11.0 kN for ϕ = 0.9 - satisfactory.

B Brace Member to Column End Connection:

Performance of end connections is to be verified by test, in accordance with Clause 4.1 of DG 8.3 [3].

C Comment:

If the upright frame was designed as a CBF to NZS3404: Part 2[5] the following would apply, assuming a Category 2 structure:

- From Clause 12.12.2, bracing slenderness not greater than $120/\sqrt{F_y/250}$ = 120 for F_y = 250 MPa.
- From Table 12.8, with 4 storeys of bracing;
  S = 2.6 or 3.0 depending on bracing slenderness.
- Braces must be Category 1 or 2 members.
- End connections must develop overstrength tensile brace forces.

It is apparent that design as a CBF to NZS3404 would require a significantly larger brace section than is current practice in upright frame construction. Similarly, the detailing requirements of NZS3404: Part 2[5] for EBF's would require the use of sections not typically used in pallet rack structures.

In design of upright frames, it would appear that the more economical designs will be obtained by:-

(i) Testing, and/or;

(ii) Frame designed for rocking response [1].

A5.9 Other Design Checks

In addition to the design checks detailed in the preceding sections, a typical drive-in rack design should include checks of items as follows:-

(a) Holding down details (for shear and tension) and adequacy of supporting structure;

(b) Combined axial and bending stresses in top tie (transverse and longitudinal) members, and connections to columns;

(c) Adequacy of pallet beams and column brackets (normally standard details which may be rated for given loading and span);

(d) Bending moment in exterior columns due to eccentric loading from pallets (including earthquake effects);

(e) Bending in columns due to "hydraulic" loading, where the nature of product and method of storage renders this a possibility. Means for reducing possible "hydraulic" loading effects should be considered;

(f) Clearances between the rack structure and the enclosing building (including building columns) should be provided to comply with Clause 3.8.2 of NZS4203 [4].

APPENDIX 6

SCHEDULE OF ESSENTIAL INFORMATION TO BE COMPLETED BY RACK PURCHASERS/SPECIFIERS

(reproduced from Appendix D of DG 8.3 [3])

SEISMIC DESIGN STANDARDS FOR INDUSTRIAL STEEL STORAGE RACKS

Appendix 'D' SCHEDULE OF ESSENTIAL INFORMATION TO BE COMPLETED BY RACK PURCHASERS/SPECIFIERS

This form should be completed by prospective purchasers intending to instal storage racks in new, or existing buildings. Information provided will be used by the rack manufacturer in preparing the structural design calculation for the storage rack.

A: GENERAL

A1 Nominated Industry Standard (Clause 1.2.1)

....

A2 General materials handling requirements (Purchaser to attach outline indicating sketch forklift usage and brief specification).

....

....

A3 Rack Type Required. (Classify as : Drive In, Drive Thru, Standard, Live Storage, etc.)

A4 Name of Local and/or Regulating Authority (for Building Permit purposes)

....

B: DESCRIPTION OF PRODUCTS TO BE STORED

B1 Product Type (list)

....

....

B2 Nature of Storage

(i) On Pallets?...................

Nominate sizes, type, stacked height

....

(ii) Maximum specified pallet load

....................kg/pallet

(iii) Minumum clear height between pallet supports:

....

(iv) Is block stack storage anticipated (without pallets

....

If yes, state products, type and method of stack

....

(v) Do any products constitute unusually high risk to life, or property, e.g. Acids, Dangerous goods, etc.

....

B3 What is operating temperature (°C)

....

C: DETAILS OF ENCLOSING BUILDING

NOTE: Purchasers to obtain professional Structural Engineering advice when completing this section.

C1 Is Building new or existing ....

C2 Name of Designer.........................

Date constructed.........................

C3 Was/Is building constructed for storage rack usage?

....

C4 Is rack installation located on ground bearing floorslab, or on an elevated floor in the building (then complete C6, or C7 as appropriate....................

C5 Is building superstructure capable of resisting lateral earthquake loads from storage racks. (Provide details, and basis of design........................

....

C6 <u>For storage racks on ground bearing floorslab</u>

(i) Permissible Gross Floor loading on slab (with corresponding settlement expected in foundation subsoils)
..............................kN/m²

(ii) Maximum point load capacity of floorslab (unfactored) under following contact pressures :

100mm square side.................kN

300mm sqwuare side................kN

(iii) Structural description of floorslab and wall skirtings (if any) including reinforcement. (Typical information to include slab structure, size and location of reinforcement, structural joints, existence of insulation, etc.)

....

....

....

....

(v) Is floorslab/foundation specifically designed to resist uplift loads e.g. from bracing anchorages ? (Clause C3.4.1)
If so, provide details of anchorage locations, and ultimate capacities

....

....

(vi) Does owner/specifier agree to use of limited floor uplift as mechanism for bracing anchorage (Clause 5.2.1).

C7: For storage racks on elevated floors in buildings

(i) Permissible gross floor load on suspended slabs, and supporting structure......................kN/m^2

(ii) Maximum point load capacity floorslab (unfactored) under following contact pressures :

100mm square side ....kN

300mm square side ....kN

(iii) General seismic design basis for enclosing building (for use with NZS 4203 Clause 3.4.9)

....

....

C8: Assumed deformation of Building Superstructure (at roof/ceiling level) under earthquake loading. (Ref: N.Z.S. 4203 Clause 3.8.2)

D: ANCILLARY FEATURES

D1 Are ancillary fittings to be supported from the racks e.g. sprinkler pipework, fittings, lifting equipment. (Provide details)

....

....

F: ONSITE FABRICATION

F1 What length of time at normal temperature will be available for rack installation (particularly site welding).

....

G: OTHER SPECIAL REQUIREMENTS

....

....

....

NAME OF PURCHASER/SPECIFIER

....

....

NAME OF MANUFACTURER

....

DATE ....