NOTES FOR DESIGNERS AND USERS OF PRESTRESSED PRECAST CONCRETE FLOOR SYSTEMS AND SHELL BEAMS

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DISCLAIMER AND LIMITATIONS

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This document is intended only to highlight some issues relating to the design and use of prestressed concrete floor systems. It is for the assistance of suitably qualified and experienced designers and is not intended for use by other persons. It is not intended as a complete design guide. Specific design issues should be referred to a professional engineer with appropriate experience. Advice may also be obtained from the manufacturer.

All designs and details are required to comply with the appropriate New Zealand Acts, Regulations and Standards including the Building Act, NZS 3101 Concrete Structures Standard and AS/NZS 1170 Structural Design Actions.

Continuing research will update knowledge and understanding of structural behaviour. Accepted best practice will evolve and the information given in this document may no longer be considered best practice. It is the practitioner's responsibility to ensure they remain up to date and apply best practice current at the time.

DESIGN RESPONSIBILITIES

Flooring suppliers normally offer a limited design service but this is usually restricted to gravity load capacity for the composite floor section.

It is important that the design engineer responsible for the building provides the magnitude and location of all loads to be supported by the floor. This includes all superimposed loads such as point load, wall loads, wind loads, snow loads, bracing loads etc. The floor designer can not be responsible for deriving loads. All loads for which the floor is designed will be noted on the supplier's layout drawings and it is important for the building designer to check the completeness of these.

While precast suppliers are prepared to offer advice on use of their products, it is necessarily limited.

The building designer is responsible for overall design of the building. This includes the performance of the floor as a diaphragm, and therefore must extend to the topping concrete strength, thickness and reinforcing. It must also cover the connection of the diaphragm to the supports and load resisting elements. To ensure performance of the diaphragm under seismic action, the detailing of the support and its connections to the floor is critical.

Seismic drift in ductile frame buildings can cause support rotation, and beam elongation may cause reduction in seating.

These considerations are beyond the scope of precast concrete manufacturers.

Shell beams are only a precast shell becoming permanent formwork while providing some level of reinforcing within the precast shell. The precast shell becomes just one component of a structural system and for this reason it is inappropriate for precast concrete suppliers to provide any design service for shell beams

Design of temporary propping is a specialist area and is not a service offered by concrete precasters. Precasters are able to supply component weights to assist propping designers.

Floor suppliers will normally indicate a minimum amount of temporary propping (ie maximum spacing of temporary supports) required for flat slab floors or rib and infill systems but this is only to cover the ability of the floor components to span under normal construction loads. If there are additional loads on the floor during construction the floor supplier must be notified of that and closer prop spacing or other precautions may be required. The precast supplier will not be responsible for the design of the actual temporary propping system.

MANUFACTURERS' LOAD SPAN TABLES

Load span tables should be used with caution. They give the maximum possible live load that can be carried using maximum prestress for each product type, and assume typical floor use and detailing. Floors are custom designed for each location and are supplied with only sufficient prestress and strength to carry the design loads given at the time of manufacture and this is typically less than the maximum that can be used. Using floors near the limit of their load span capabilities is not recommended to avoid issues such as excessive liveliness, deflections or precamber.

Floors are normally designed on a simply supported basis but detailed with continuity reinforcing at the supports. Floors with a "T" shaped cross section, e.g. double tees and rib and infill systems, have a limited compression zone available for developing negative moment at the supports, and that zone is already pre compressed due to prestressing. Design on a continuous basis is restricted by the limited compression zone available and allowable moment redistribution.

The continuity reinforcing at the supports that does not meet the code requirements for full continuity, will still significantly affect the performance of the finished floor. Floors detailed as simply supported must have special consideration given to the seating detail and the deflection characteristics. The floors sizes and types indicated from the load span tables are for typical detailing and may not be appropriate if nominal end restraint is not available. Check with the supplier to confirm suitability where a nominal level of end restraint is not available or where particular performance criteria such as low levels of deflection are wanted.

A truly simply supported element has 5 times the deflection of a fully continuous element of the same dimensions under normal uniform loading. To achieve the same deflection, a rectangular section of constant width would require a 70% increase in depth. A 250 mm deep fully continuous rectangular member would need to increase to 425 mm deep to have the same deflection if it was simply supported.

POINT LOADS

It is important to remember these are floor systems, and while they can support a certain level of concentrated load (wheel loads, point loads, wall loads etc) their ability to act as beams is limited and they are not intended to replace major loadbearing elements, such as beams, walls, highway bridge decks etc.

It is not possible to assume a point load is within the floor capacity by relating it to a distributed load. This applies particularly to wheel loads. A large point load close to the support may not be shared with adjacent units, resulting in higher than anticipated shear demand on individual elements.

Load sharing of point loads located near mid span may enable them to be supported by more than one unit when considering critical bending moments.

Load sharing may not be available for point loads located near the support where shear stresses may be critical. This is because shear forces close to the support may not cause sufficient deformation to transfer load to adjacent units before a brittle failure occurs in the unit directly under the load.

EXISTING FLOORS (Design checks)

Designers checking the capacity of existing floors should not assume a previously constructed floor has the load capacity given in load span tables. Each floor is specifically designed and only the moment and shear capacities required for the design loads specified in the contract documents may have been provided. To check the ability of an existing floor to support a different load, designers should check the design load originally specified and ensure any revised load does not exceed the bending or shear capacity of any element.

The original floor supplier may have records of past jobs, but searching archives involves a cost with no guarantee of locating anything of relevance.

SELECTION OF FLOOR TYPE

Selection of the appropriate floor type from those available depends on a range of factors including geographical location, building shape and size, building design, seismic requirements, site access, lifting facilities, available skills, loads, spans, fire ratings, acoustic requirements, end user expectations, exposure conditions, personal preference and previous experience of owners, designers, developers or clients.

Larger buildings with good access, reasonable transport distances, suitable crane facilities, and large rectangular bays with medium to long spans will favour large component systems such as Hollowcore or double tees.

Drilled penetrations for services or notches formed by concrete cutting can be accommodated in either floor type but cast in penetrations cannot be formed in hollowcore flooring units.

Flat slabs provide economical solutions up to 6 metre spans although they can be used for longer spans. They provide a solid slab with a flat underside (longitudinal joints at 1.2 or in some cases 2.4 metre centres), and are popular where the off steel form finish to the underside is an acceptable surface for a ceiling, with or without further treatment.

Typical flat slabs are 75 mm thick that act compositely with a range of topping thicknesses. Increasing the topping thickness requires solid concrete over the full area and other floor types may be more economical where increased thickness is appropriate. With longer spans consideration should be given to live load deflections, self weight and temporary construction propping requirements. Floor vibration may also be a client consideration.

Rib and infill floors are versatile and widely available. They are light, economic to transport (they are exported to the Pacific Islands), well suited to fit irregular shapes, and can easily accommodate services, penetrations etc. They use the lightest individual components which makes them particularly suitable where access for large cranes are difficult. Different infills such as dressed timber can be used.

If in doubt, discuss your particular application with a manufacturer.

SELECTION OF FLOOR DEPTH

Often the thinnest floor is not the most economical.

Increasing the floor thickness may only require a small amount of additional concrete to form a deeper rib or web. The thicker floor may require less prestress and shear reinforcing which can go some way towards offsetting the cost of the extra concrete in the precast unit.

The thicker floor will provide better performance characteristics in respect of deflections and perceived "liveliness."

With unpropped systems such as double tees, selecting the shallowest floor can lead to camber problems. Shallower sections require more prestress to carry the load, causing a higher eccentric moment, which is resisted by a section with a lower moment of inertia. Combine higher eccentric moment with lower moment of inertia and the lighter self-weight reducing the gravity deflection and there are three factors increasing the camber. Add creep into the equation and the problem just gets worse.

Selecting the thinnest possible floor may increase construction costs as the builder accommodates higher cambers, which require adjustment to support levels, and extra topping concrete at the supports.

FLOORS EXPOSED TO WEATHER

Concrete floors are typically used in enclosed buildings (A1 exposure classification) rather than exposed to the weather. Where a floor forms the top level of an enclosed structure, the top surface will normally be waterproofed (see "WATERPROOFING")

While concrete has a high level of durability and suspended precast floors can be detailed for a long design life in exposed conditions, this requires the floor topping to be of suitable thickness with appropriate levels of reinforcing protected by adequate thickness of cover concrete of suitable quality, rather than what is typically used for flooring in enclosed interior conditions.

Suppliers load / span tables are based on prestressing tendons located with concrete cover for typical indoor application. Other exposure may require the location of the tendons or other reinforcing to be altered or different stress level to be used thereby changing the load carrying capacity and possibly self weight of the floor. Check with the manufacturer.

The prestressed elements themselves can be designed to remain uncracked while in service, and the high strength concrete used in their manufacture, combined with the control of concrete cover inherent with pretension processes gives superior durability characteristics for exposed conditions.

Floors where the top surface is exposed can have significant temperature variation over their depth. The top surface may be covered with black waterproofing material resulting in high absorption of solar heat, while the underside may be shaded and ventilated as in a carpark, or temperature controlled as in an air conditioned office building.

The top surface temperature of a floor exposed to the sun varies relative to the bottom on a daily cycle. The temperature gradient through the depth of the floor results in different thermal expansion throughout the floor depth causing the floor to go through daily hog or sag cycles, which in turn causes a rotation and sliding at the supports. Daily fluctuations of 30 mm to the level at midspan of a completed floor have been measured. Depending on the detailing, the rotation at the support can result in sliding between the bottom of the floor units and its support. This has caused severe damage to the structural components at the interface with serious consequences in several instances. Designers should keep this possibility in mind and adopt appropriate detailing. This may involve low friction bearing strips, embedded steel plates or other solutions but the effect on floor performance should be considered and appropriate floor types and sizes used.

Refer to experienced engineers or your manufacturer for advice.

WATERPROOFING

Where the top surface of a concrete floor is exposed to the weather, it is important that appropriate waterproofing details are used. This is to prevent water entering the interior of the building, and also to prevent water penetrating into the floor structure.

Waterproof additives in the topping concrete have been found to be ineffective. Water transmission through well-compacted concrete is low without additives, but thin concrete floor toppings are subject to shrinkage cracks and flexural cracks. These cracks alter in width with time, temperature, load etc and waterproofing the concrete does not waterproof the cracks.

Due to the fluctuations of crack widths, painted on membranes, sealing or filling individual cracks after they appear, have generally been ineffective in the long term.

Shrinkage compensating concrete has had limited success in providing a waterproof topping. It may compensate for (but not eliminate) shrinkage but has limited effect on flexural cracks and temperature effects. Shrinkage compensation causes the concrete to expand during the setting process, but the shrinkage occurs at a different stage. It may not be effective unless the initial expansion is restrained, and is sufficient to compensate for all long term shrinkage.

Waterproofing is best achieved using a separate flexible membrane, which is properly detailed and has the ability to withstand crack width fluctuations. Expert advice should be sought to source a suitable membrane, and also to ensure proper detailing. Floor suppliers are not membrane specialists.

Detailing of the membrane must ensure there are no locations where water can get behind the membrane.

Membranes have lifted due to water vapour trapped under the membrane being heated by the sun and venting is sometimes used to overcome this.

Waterproofing membranes may require protection from foot traffic or other uses.

The floor topping should be detailed with adequate weathering steps and falls to ensure the water drains away and does not accumulate on the floor surface. Steps and falls within the floor topping must be proportioned to accommodate expected deflections of the floor structure. This includes deflection on removal of any props directly supporting the floor or its supporting beams during construction, deflection of beams or supporting structure, deflections due to loads, shrinkage, creep etc. Longer spans have disproportionately greater deflection.

Because of the variables affecting deflections, deflection predictions are only estimates and are subject to large variations, particularly over time. It is difficult to accurately compensate for floor deflections during casting of the floor topping. For these reasons, steps and falls must be generous and allow for inaccuracies and variations in assessing and compensating for the floor deflections. Generous tolerances are also required to allow for varying skill levels of concrete placers and finishers.

SHELL BEAMS

Precast shell beams are open at the top to allow a reinforcing cage to be placed, and the core filled with in situ concrete on site. The precast shell is prestressed at the bottom and this prestress can be used to supplement the midspan bending moment capacity of the completed beam. The units have an off steel mould finish to the external face and roughened internal surface to provide bond with the in situ concrete core.

Shell beams are manufactured in standard moulds, of 400mm, and 600mm width. Moulds currently in use enable shell beams to be cast with side heights up to 600mm. Each side can be a different height and step to different heights along the length of the beam. Sloping side heights are difficult to form accurately. The minimum practical side height is 300mm and although lower sides have been used, they cause difficulty with manufacture and ensuring sufficient surface for the in situ core to bond with the shell to achieve composite action.

Standard side heights are 400 and 600 mm. Different side heights are formed freehand within a narrow side to the mould and will not have the accuracy obtained with the standard side heights.

Extreme differences in side height will result in the section centroid moving horizontally and the prestressing becoming eccentric horizontally. This will cause horizontal bowing of the beam in addition to the vertical hog and may cause some construction difficulty. This is accentuated as one side height reduces below 300 mm.

The shell beam becomes permanent formwork while providing some supplemental midspan moment capacity from its prestress. It requires a reinforcing cage in its in situ core to give continuity moments at the supports, and to provide shear capacity. The reinforcing cage continues over the supports and for the full length of the beam and provides some of the mid span moment capacity. The beam shells are too thin to accommodate shear reinforcing and all shear reinforcing must be provided within the in situ core.

Once the in situ core is cast, the shell can act compositely with the core. This is dependent on adequate bond between the shell and the in situ core. If the shell beam sides are very low, the area available for bonding may be inadequate, especially where there are high floor loads seating onto the beam shell, and it may be necessary to provide additional reinforcing to bond the shell to the core.

Reinforcing can be provided to tie the shell to the core where there is concern about the available bond, but manufacturing processes limit the ability to incorporate additional reinforcing in shell beams. If additional reinforcing is likely to be required, it should be discussed with the manufacturer early in the design process.

Shell beam support levels may be lowered to accommodate shell beam camber and the camber of flooring supported on the shell beams. This needs to be considered before construction of the supports is completed.

Propping during construction

Shell beams are normally propped during construction eliminating the need to calculate the stresses and deflections in the shell at the various construction stages. With the shell being prestressed and propped, it is likely to remain uncracked under service loads.

Shell beams can be designed to be unpropped for limited spans and loads, but this should be discussed with the manufacturer early in the design process to determine if it is feasible. It should also be highlighted on the tender drawings as it is not a normal requirement for shell beam supply, and may be missed when competitive tenders are obtained. Care must be taken to ensure the seating is adequate and suitably prepared. The cover concrete at the top of cast insitu columns may not provide suitably level surfaces and bearing length during construction.

The narrow sides of shell beams limit the available compression zone for bending and this, together with construction loads, restricts the distance that shell beams can span without propping during construction.

Propping design

Failure of temporary propping to beams is a particular hazard that has potential for fatal consequences. Design of temporary propping must be carried out by a suitably experienced and qualified person. It is not an area in which to take short cuts and a conservative approach should always be taken.

Maximum spacing of temporary props depends on the shell beam design, and also on loads applied during construction. This requires the builder to coordinate between different specialists to ensure the maximum prop spacing is appropriate for the builders proposed construction method.

A typical minimum would be props within 300 mm of each end, and at a maximum of 2,400 centres between, with props under both edges to prevent the beam rolling. Propping close to each end is recommended to guard against poor seating. Closer spacing is frequently required.

Construction loads on the shell beam include the shell beam self weight, its core concrete and reinforcing as well as loads from any floor units seating onto the shell beam and any construction loads on the floors. The amount of load from the floors is also affected by temporary propping of the floors. In some cases the floor units seating onto the shell beam are propped adjacent to the shell beam thereby removing load from the shell beam during construction and reducing the propping requirements for the shell beam.

Maximum spacing of temporary propping along the shell beam depends on the ability of the beam shell to span while subject to the construction loads. Practical limitations on prop capacity may result in closer spacing to cope with the level of construction load

Propping will typically follow the natural camber of the beam and levels of the propping may need adjusting to ensure the load is distributed evenly between the props.

Setting centre props lower to allow the beam to settle and reduce its camber during placing of the floor and in situ concrete should only be done after careful consideration of the effects on the stress levels in the shell beam. It is also important that allowance is made for the increased load to the props near the supports.

Temporary construction propping to shell beams and the flooring they support should be detailed to control the tendency of the beam to roll when the floor units are placed on one side only during construction.

In some cases the floor systems are propped adjacent to the shell beam to avoid the floors loading the shell beam during construction and remove their tendency to cause the beam to roll.

As always, temporary propping design should consider the load path to the ground and all intermediate stages on the way to the ground. It should also consider bearing stresses and settlement or deflection and the effect on load redistribution.

Beam design

Because the precast shell is only a part of the beam, which itself may be only one part of a structural system, it is inappropriate for the design of the shell and its reinforcing to be carried out by anyone other than the design engineer responsible for the structure. This is different from precast floor systems for which manufacturers normally offer a design service to carry gravity loads because the floor can be treated as an isolated element, whereas the beam shells cannot.

For this reason suppliers do not offer a design service although some may provide limited capacity figures, which can be used as a guide by appropriately qualified designers.

BEARING LENGTHS AND SEATING OF FLOOR UNITS

The suitability of the end seating details is the responsibility of the structural designer, and the following comments must be interpreted and applied by an appropriately qualified and experienced professional.

It is important that the designer clearly specifies and details the seating requirements and the connections between the structure and the floor. Design of the supports must allow for the required seating details to achieve Code compliance and have provision to accommodate construction tolerances.

Detailing must allow for the required seating to be achieved AFTER allowing for tolerances.

In many cases, seating of flooring units onto unreinforced cover concrete is inappropriate.

Low friction bearing strips should be used for seating Hollowcore units to accommodate movement without damaging either surface at the bearing interface.

With tees, low friction bearing strips should be used. Flange support systems concentrate the bearing stresses and in some cases it may be appropriate to have embedded steel bearing surfaces. Refer to your supplier for details.

NZS3101:2006 Clause 18.7 covers connection and bearing design and Clause 16.3 covers bearing stress

Seating length requirements after tolerances are 50 mm for flat slabs to 9,000 span, 75 mm for other floor systems to 13,500 span, then span/180.

It is important to ensure support is not lost during a design seismic event. Collapse of a floor due to loss of support, may result in failure of any floors below it with catastrophic consequences. Where ductility may occur under seismic attack, the combination of beam elongation due to formation of plastic hinges and rotation of the support together with shrinkage and creep and allowance for tolerances may make it necessary to detail a greater width of bearing ledge than used in earlier practise.

The possibility of moment reversal and the effect of support rotation under seismic attack should be considered.

Support rotation and beam elongation under seismic attack can cause loss of end restraint moment capacity, but the primary consideration will be avoidance of collapse.

Because of the variety of details and applications, only some general principles can be covered here. It is important to comply with code requirements and the latest information.

Where the floors are continuous over supports and a restraint moment is being developed, providing there is no possibility of the moment being reversed at the support, the loads may be capable of being transferred to the supports by shear through compression zones, instead of by bearing. In these cases structural integrity can be achieved without direct bearing through the seating.

Where there is no continuity at the supports, or where moment reversal may occur under seismic attack, the situation is different.

If the end of the floor unit is not cast into the support, with well compacted concrete against the vertical end of the precast unit, and negative reinforcing from saddle bars or starters provided, then very different conditions apply because restraint moments can not be reliably developed. Deflections, floor performance, bearing surfaces all need particular consideration and thicker floors may be required.

Consideration should be given to both daily and seasonal temperature variations, as well as changes in temperature differences throughout the depth of the floor

If sliding of the precast unit on its support may occur, from temperature variations or other actions, the support detail needs to be able to accommodate that movement without loss of integrity.

Consideration should be given to whether the movements are likely to be concentrated in a few locations. This is more likely to occur where there are thermal movement joints in the main structure, or where the ends of the floor are not built in.

Temperature differences through the depth of a floor that change on a daily basis, (such as the top floor of a building subject to daily temperature variations due to solar heat gain while the underside is not subject to the same temperature fluctuations), may cause daily rotation and sliding at the support. This can be particularly damaging if the seating is not suitably detailed to accommodate it.

Flat Slab Flooring Units

These are generally used for shorter spans requiring lower shear forces to be transferred to the supports, and as they have bearing over their whole width, bearing stresses are low. Minimum recommended seating is 50 mm onto concrete or steel and 65 mm onto blockwork to ensure the bearing extends past the face shell.

Rib and Infill Floors

Previous practice complying with the 1995 version of the code was to apply the same recommendations to moderate spans and loads where only limited seismic deformations were expected. That was 65 mm seating onto blockwork to ensure the bearing is past the face shell, and 50 mm seating onto concrete or steel. The 2006 code now requires seating after tolerances of 75 mm to 13,500 span and span/180 over that.

Flange Supported Double Tees

Flange supports may transfer load through small bearing areas. Low friction bearing strips should be used, but in some cases it will be necessary to provide steel bearing plates or similar to limit the bearing stresses on the support and accommodate movements. Refer to local manufacturers for flange support details available. Some have supports to accommodate either 75 mm or 135 mm seating. Refer to “DOUBLE TEES” for further comment.

Web Supported Double Tees

Refer to “DOUBLE TEES” for further comment, but generally not less than 75 mm seating length should be used with low friction bearing strips or steel bearing plates.

Hollowcore Units

The use of low friction bearing strips is a code requirement for most applications where the units are supported on hard surfaces rather than the support being cast at the same time as the topping.

For buildings which may be subject to significant level of seismic drift, details given in the latest revisions to NZS3101:Part 2:2006 or other appropriate details should be used.

The reinforced detail (NZS3101:Part 2:2006 Figure C18.5) involves seating on a low friction bearing strip and breaking out two cores in each unit, then placing a round (not deformed) bar HR12 with a hooked end in the bottom of those two cores, and filling those cores with concrete as the topping is placed. These are single bars, not formed into “hairpin” or “paperclip” shape. This detail was developed after extensive testing at Canterbury University to accommodate high levels of support deformation from interstorey drift in ductile moment resisting frames under extreme seismic attack.

The reinforced detail maintains the continuity of the floor with reduced deflections etc and provides greater tie in to the supporting structure.

There is provision for use of other details or seating lengths that can be justified by analysis or testing. Other details may be acceptable for lower expected levels of seismic deformation.

The backing detail (NZS3101:Part 2:2006 Figure C18.4) was deleted by Amendment 2. This had been tested prior to inclusion in the original publication, and while it is no longer the Code preferred solution, it may be suitable for some applications.

Testing has indicated higher levels of core reinforcing at the ends can cause problems where the reinforcing terminates if the supports rotate under seismic attack.

SEATING RIBS INTO POCKETS OR ONTO REBATES

End restraint becomes a major consideration where it is intended to seat flooring units into pockets or onto rebates cast into a beam, or use a similar detail such as seating onto a steel angle fixed to the side of a beam or wall. If the end of the flooring unit is not fully encased in well compacted concrete, the end of the flooring unit is not restrained from moving and this will significantly increase deflections and cause degradation of the support through sliding.

It will be necessary to increase the floor depth significantly (perhaps by 30% to 60%) and take steps to protect the interface between the precast element and its seating, unless appropriate measures are taken to ensure end fixity is achieved.

End restraint can be obtained by ensuring there is sufficient top reinforcing over the supports and the ends of the unit are cast into concrete at the supports at the same time as the topping is cast.

When seated into preformed pockets or onto already cast ledges or similar, the space between the vertical end of the precast element and the supporting beam should be filled with a suitable high strength well compacted grout before the temporary propping is removed. Although this is easy to specify, there have been instances where the grouting has been inadequate and the work totally ineffective, leading to client dissatisfaction with their building. It is important to provide top reinforcing and to achieve a compression zone that does not require movement to develop compression and a restraint moment.

PROPPED SYSTEMS SUCH AS RIB AND INFILL AND FLAT SLABS

Where a precast element is propped during construction and is detailed with reinforcing to resist negative bending at the supports, the soffit of the composite floor is under compression at the support and its performance under gravity load is analogous to a fully continuous cast in situ monolithic section. In this case the junction between the precast element and the in situ concrete is little different to a construction joint in a cast in situ floor, although the location of the construction joint may be unusual.

For structures such as low rise shear wall buildings which are not subject to significant levels of seismic drift, provided the moment at the support will not be reversed, the integrity of the complete floor is not sensitive to the amount of end bearing or lack of it.

Shear transfer from the floor to the support will be through compression zones that remain in compression, and not by bearing through horizontal surfaces that may move relative to each other. This only applies to propped floor construction suitably reinforced and detailed at the supports, and is typically limited to rib and infill or precast flat slab floors in low rise shear wall structures with limited seismic drift. All detailing should be in accordance with the appropriate design codes.

Rib and infill floors have been detailed to be supported from upstand beams where no endbearing has been possible because the soffit of the supporting beam is at the same level as the soffit of the floor. This has been done by leaving the prestressing tendons protruding from the element and bending them up into the upstand beam. Bending the prestressing tendons may not be easy and sometimes heat has been applied. While it is not considered good practice to apply heat to prestressing tendons because it substantially weakens them, once they are at a distance away from the precast element, their full strength may no longer be required, and their reduced strength may be more than adequate to transfer the vertical load into the upstand support.

Where the end seating detail of the precast element does not meet code requirements for length of seating, it may be possible to transfer the vertical load from the composite floor into the supporting structure using the shear friction provisions of the code. This requires transfer of the shear forces from the precast element through the in situ topping and into the support. Bond of the topping to the precast element should be considered and reinforcing ties may be required to connect the topping and precast unit. Where the lack of seating is intended by design, it may be possible to leave prestressing tendons protruding from the end of the element to meet the shear friction requirements of the code in which case transfer of forces through the topping is not an issue.

Prestressed ribs can be manufactured with shear stirrups at close centres at each end which helps to provide proper confinement of shear friction reinforcing for a statically determinant solution, and reduces the possibility of separation between the in situ topping and the precast rib where shear friction reinforcing is used in the topping. If this approach is required, it is important to discuss it with your local manufacturer.

DOUBLE TEES

Double tees may be flange supported, web seated without the ends cast in, web seated with cast in ends or with reduced depth web at the seating, sometimes referred to as “Dapped Ends”. Because they are not normally propped during construction, restraint moment effects are either not available, or only developed by the applied load component, double tees are normally deeper than other floor types for the same span and load conditions to compensate for the lack of end restraint effect on deflections.

Flange supported tees have no continuity restraint moment available. A conservative approach is required in selecting unit depths of flange supported tees to limit deflections and “liveliness”. It is not appropriate to use units near their recommended load / span limits.

Flange supported tees were previously detailed with 90 mm flange extension to accommodate 75 seating and 15 mm clearance at each end. Flange seating extension of greater than 90 mm is available to meet the current code requirements, and also for seating onto steel beams where it is desired to locate the load close to the web.

If the building is a ductile frame design, the clearance between the end of the tee web and the vertical side of the support may be important to accommodate seismic drift, and in such cases the contractor needs to be made aware of the importance of maintaining the end clearances during construction.

Where seismic drift would cause rotation of the supports relative to the floor, it may be necessary to construct the tees with the end of the web sloping back to accommodate rotation of the support.

There are different flange support systems available. Some are an embedded steel assembly, which is statically determinant, and all critical member sizes, location and alignment can be checked after delivery to site. These systems of flange support are robust and not easily damaged during normal handling. They are not brittle and minor yielding will not reduce their strength. They can accommodate distortions within the precast unit and shrinkage or movements within the building structure without loss of strength. They do not depend on the concrete topping or any after delivery work to achieve their full strength.

Flange supports using “pigtail” or “loop bar” type reinforcing are suitable for use in many applications. They have been in use as a standard detail for over 30 years without problems and have been subject to testing on many occasions. Despite a lack of past problems, concerns have recently been expressed regarding their ability to accommodate end rotations resulting from seismic drift. Further investigation of this is underway in the later part of 2008 and will continue in 2009.

Refer to suppliers for further information on different flange support systems and available details and ongoing investigations.

The different types of flange supports concentrate bearing stresses differently. As a minimum, low friction bearing strips should be used, but high bearing stresses may require steel seating cast into the support, or packers used to spread the bearing load.

The short flange extension used for flange supported tees may cause unacceptable torsion or other effects in the supporting beams, particularly where seated onto only one side of steel members. In these cases extended flange supports are available to transfer the load from the floor into the support closer to the centre of the supporting member, but they have reduced capacities due the increase in bending moment in the top bar or flange extension. These may extend 150 mm past the end of the tee providing up to 135 mm of seating with 15 mm clearance.

Where units are flange mounted onto steel supports, it is possible to cut the flange back between the webs to accommodate studs on top of the steel member, but coordination between trades and accurate setting out is essential to ensure the stud positions do not clash with the flange supports.

Web seated tees without the ends cast in will not have the ability to develop continuity restraint moments at the supports, and again a deeper unit may be appropriate.

This situation occurs where a supporting beam is cast to the seating level of the tee or higher or where a steel member (such as an angle) is attached to the side of a beam or wall to provide seating. There is no reliable means of providing restraint moment and for this reason it is important to accommodate movement between the precast unit and the support with low friction bearing strips or steel bearing plates.

Accommodation of construction and manufacturing tolerances and misalignment between the precast unit and its support should be considered when detailing seating.

Where web seated tees without the ends built in are used, the concrete support may be reinforced with a steel angle to spread the load and reduce the bearing stresses, and the precast unit may have a steel shoe embedded at the ends. These units should not be seated onto the cover concrete of a concrete supporting beam. Low friction bearing medium may be used but it is important to ensure it can withstand the expected bearing stresses.

Web mounted tees with the ends built in. The added restraint available at the ends will affect the floor performance in terms of deflection, and also the perceived “liveliness”. Because double tees are typically unpropped during construction, the level of restraint moment is low, only being developed during application of superimposed loads after the topping has been cast so the full benefit of continuity on floor performance will not be available.

These floors are sometimes supported on cover concrete, but this is only appropriate for short spans with low loads in buildings not designed using ductile seismic frames.

It is possible to leave prestressing tendons extending from the ends of the tees to cast into the support. This can be helpful seating onto block walls, but creates difficulties with placing beam reinforcing when seating onto beams.

Reduced depth web support or “Dapped Ends”. The end of the tee web is notched to reduce the depth locally so the tee is seated part way between the underside of the web and the underside of the flange. This is used to reduce the combined overall height of the supporting beam and floor combination, and is part way between a web mounted and flange supported detail. This detail has been used many times but because of the number of variables, there do not tend to be industry wide standard details. Each instance requires specific design and should be discussed with precast suppliers prior to use to confirm practicality of details. Consideration should be given to development of reinforcing, shear transfer through non prestressed sections, and bearing stresses.

Single tees Where a bay width is such that a single tee would fill the remaining space, it may be preferable to use two or more narrower double tees. The single tee requires cradles for storage and transport, and can be subject to buckling during lifting if the slings from a single crane hook impart a longitudinal force on the tee. There is no cost advantage in using a single tee because it occupies a full width mould and requires special handling. Beware of different cambers and possible sideways deflection resulting from changes to section properties.

Narrow units Tees are manufactured in long moulds and non standard units such as narrow or single tees will have the same prestress as the standard units being manufactured at the same time. The effect of the prestress on the different cross section will alter the camber, typically increasing it, sometimes significantly. In some cases it may be preferable and cheaper to make up a bay width by boxing a strip between standard units rather than using narrow or non standard units. Narrow double tees (as distinct from single tees) do not generally have sufficient eccentricity to cause horizontal deflection problems.

TEE DEPTH, CAMBER AND COST

Deeper floors will carry longer spans or higher loads. To choose a double tee unit one or two sizes deeper costs little more. This is because there is little additional concrete added to the double tee legs and the cost of this extra concrete may be partially offset by a reduction in prestress and shear reinforcing. It can also reduce the finishing costs by providing units with less camber. When selecting double tees for a load span combination, the shallowest possible precast unit may not provide the most economical finished floor, nor the most suitable floor in terms of performance (deflections, vibration etc).

Selecting the shallowest floor can lead to camber problems. Shallower lighter sections require more prestress to carry the load thus causing a higher eccentric moment that is resisted by a section with a lower moment of inertia. Combine a higher eccentric moment with a lower moment of inertia and the lighter self weight reducing the gravity deflection, and there are three components increasing the camber. More creep and higher levels of stress increase the problem.

Camber in the tees requires the support levels to be adjusted and extra topping concrete is required to maintain minimum thickness at midspan.

Increasing the tee depth for a given load span combination requires only a small increase in the self weight and amount of concrete because only the narrow webs are increased in depth, but the reduced amount of prestress and shear reinforcing, can go some way to offsetting the increase in concrete cost. The reduced camber will reduce the average amount of topping concrete required to maintain the specified minimum.

Increasing the depth of the precast unit can result in lower finished costs while improving the performance of the finished floor.

PARTIAL PRESTRESSING TO CONTROL CAMBER OF TEES

Partial prestressing is a means permitted by the design code to reduce the level of prestress. This technique can be used to reduce the amount of camber in double tees.

One major reason why prestressed precast floors are economical is that much thinner sections can be used. They remain uncracked under typical working load whereas reinforced non prestressed sections of the same dimensions would be expected to crack. Cracked sections need to be much deeper to achieve the same moment of inertia, otherwise they result in substantially increased deflections.

Partial prestressing allows for soffit tensions to be increased to levels that would normally result in cracking, but with restrictions to the level of tension, or the amount the soffit tension will vary under application of the live load.

While traditional prestressed floor construction has been shown to provide very robust finished floors, some factors contributing to this are not present in partial prestressed tees. For this reason careful consideration should be given before using partial prestress design as there will be little reserve before onset of cracking and increased deflections.

Prestressed floor systems would only be expected to crack under transitory or peak live loads, if at all.

Partially prestressed systems should only crack under transient live loads but close again when the transient load is removed. If cracks remain open under long term loading, deflections and creep effects increase.

Double tees, partially prestressed to control camber, have been used successfully and been shown to perform satisfactorily in a number of major projects.

PROPPING OF TEES OR HOLLOWCORE

Large component systems such as double tees or hollowcore units are normally placed without temporary propping, but temporary propping may be used for aesthetic reasons or for structural design reasons.

In some cases, where exposure conditions require it, temporary propping may be used to ensure the soffit remains under compression to eliminate the possibility of flexural cracks during service.

Where propping is used for tees or hollowcore it <u>may</u> not be required until after the units have been placed and immediately prior to placing the concrete topping. Refer to the manufacturer or designer for advice.

Setting props to a level that is too high can cause tension cracks to occur at the top surface and progress further down.

Aesthetic Temporary propping may be used to even out the natural variations in camber where the underside is to be left exposed (for instance in car parking buildings). This is only used as spans get longer relative to the depth of unit being used. For instance 200 mm hollowcore units are unlikely to require propping for these reasons for spans less than 9 metres, depending on the degree of scrutiny the finished soffit will be subject to. Props used purely for aesthetic reasons may only be carrying a nominal load from application of the topping.

Structural The precast floor system does not have its full strength until the topping is cast and has gained strength to provide composite action. Construction loads (self weight of the floor unit, weight of the wet concrete topping, loads during placing and working of the topping) can either exceed the capacity of the bare unit to support with adequate safety, or they can result in soffit stresses that leave insufficient in reserve for the effects of the applied loads. In these cases temporary propping may provide a solution. Typically this is only for longer spans, thicker topping, high loading, or for longer spans of hollowcore units spaced apart with infills.

CALCULATION OF PROPPING EFFECTS

Consider the effects of temporary propping using superposition principals. Calculate the theoretical stresses and deflections that would occur if the bare unpropped unit remained uncracked while it was in place and supporting the load from the wet topping. Then calculate the force required to raise the unit with wet topping up to the level that the temporary propping is to be set at, and calculate the stress changes that occur. When the temporary propping is removed from the floor after the topping is up to strength, the effect is the same as applying the calculated propping force to the top surface of the completed floor. The deflections and stress changes will be reduced accordingly because they act on the composite floor section. Additionally, restraint effects from reinforcing at the supports may also be activated and reduce deflections and soffit tension.

HOLLOWCORE FLOORING SYSTEMS

The following is an extract from the Department of Building and Housing Codewords Issue 20

> The Department has no general concerns about the safety of hollowcore floors in normal service, provided the design and construction complied with all relevant requirements when the building was constructed.
>
> The Overview Report concludes that the concerns raised by the University of Canterbury test are unlikely to apply to most buildings. However, in some cases the structural performance of hollowcore floors may not be sufficient, particularly with flexible (frame, rather than shear wall) buildings.
>
> Bearing this in mind, we offer the following advice.
>
> - Owners of buildings with hollowcore floors should read the Overview Report (download a copy from www.dbh.govt.nz).

- If owners have further concerns, they should employ a suitably qualified structural engineer to review the building and hollowcore floor details.
- Remedial action should be taken if necessary The Department is encouraging the development of technical guidelines that will help structural engineers assess the performance of existing buildings with hollowcore floors and design any remedial measures needed.

Recent testing and design code changes

The Northridge earthquake in 1990's caused a well publicized failure of a hollowcore floor seated onto steel beams used in a carpark building. The connection between the hollowcore floor and the steel beam as constructed would not have met the requirements of the current New Zealand standards or those applying at the time of that earthquake.

Testing carried out at Canterbury University following this produced some unexpected results. This testing involved a ductile moment resisting reinforced concrete frame with a hollowcore floor and during the testing a number of problems became apparent. These caused concern and a technical advisory group was established to recommend directions for future research and testing. The problems only occurred as rotations at the support were increased to correspond to very high levels of interstorey drift. The test was intended to replicate seismic attack on a ductile frame building with units spanning past intermediate columns.

Subsequent testing has shown the problems occurring in that test can be overcome with relatively minor changes to detailing, and the results of this testing have been incorporated in details used in the concrete code commentary NZS3101:Part 2:2006 and amendments. Ongoing testing and research may show other details will perform satisfactorily.

The detail in figure C18.5 (Reinforced detail) involves seating on a low friction bearing strip, breaking out two cores in each unit, then placing a round (not deformed) bar HR12 with a hooked end in each core at the bottom, and filling those cores with concrete when the topping is placed. These are single bars, not formed into "hairpin" or "paperclip" shape.

The reinforced detail maintains the continuity of the floor with reduced deflections etc and provides greater tie in of the structure compared to the isolation detail.

Testing has indicated higher levels of core reinforcing at the ends can cause problems where the reinforcing terminates if the end supports rotate under seismic attack.

Hollowcore units adjacent to parallel beams may be subject to deformation incompatibility with the beam if they are deforming under seismic attack. For these cases the code requires the Hollowcore units to be spaced a minimum of 600 mm from adjacent beams using an in situ slab with span to depth ratio not less than 6 to 1. (NZS3101:2006 cl 18.6.7.2)

Enquiries regarding the damage sustained to buildings in the Northridge earthquake found that the failure in the carpark building which led to the concerns and investigations into hollowcore floor performance was an isolated occurrence. Other buildings with hollowcore floors in the area did not suffer similar damage, and there have been no particular requirements or restrictions imposed on the use of these flooring systems in the Northridge area as a result of this earthquake or that isolated failure. The seating length used in that building was significantly less than required by the New Zealand standard and the seating detail would not have complied.

Geographical limitations

Manufacture of Hollowcore units requires capital intensive plants that require a certain level of output to be economic. They have only been established in a limited number of locations.

The units require careful handling, transport and storage. They leave the factory on a truck and are not normally handled further until they are lifted from the truck to be placed into their final position in the building. Storage on the construction site or elsewhere, or trans shipping is not recommended. Geographical limitations are governed by the economics of transport and relative costs of available alternatives.

Hollowcore units are relatively expensive to transport, and the greater the distance from a manufacturing plant, the more likely it is that other systems will be more economical.

Manufacturing limitations

Hollowcore flooring units are manufactured by long run extrusion or slip form process. The only reinforcing they contain is the longitudinal prestressing. There is no transverse reinforcing or shear reinforcing cast into the units but the low water cement ratio and high levels of machine compaction produce very high strength concrete to overcome the requirement for stirrups for normal use.

The manufacturing process does not allow for departure from standard profiles for individual units. A full bed length (100 to 200 metres) of the one profile is cast and cut to length after reaching sufficient strength to transfer the forces from the prestressing tendons into the concrete. Full width square ended units without holes, notches, penetrations or any variation from the profile are produced. Any further modification such as a narrow unit, notched or angled end is carried out on the completed unit and the remainder of the unit must be disposed of as waste. The cost of a narrow unit is greater than the cost of a full width one due to the additional handling, cutting and waste disposal costs.

Different profiles such as bleacher units can be cast very economically, but there is a considerable lead time to manufacture the forming system, and a high establishment cost.

Other Hollowcore considerations

Use of hollowcore units spaced apart with timber infills between them not only reduces the cost and the weight of the floor, it also reduces the number of non standard units and enables penetrations and services to be accommodated within the timber infill section. Floors using spaced Hollowcore units available from some plants retain their fire rating and sound transmission properties due to the modified profile used to increase the concrete topping thickness over the infills.

Hollowcore units are particularly suited for large rectangular bays where a number of identical units can be supplied without cutting, notching, drilling, angled ends etc.

Because the units have no transverse or shear reinforcing, they require modification where particularly high shear forces can occur. This always applies to floors that may be subject to load from trucks. In these cases the shear created by one wheel adjacent to the support can be several times the shear generated by typical distributed loads. Increased shear strength can be obtained by breaking out the cores at each end, placing additional reinforcing in the cores, and filling the cores with concrete when the topping is placed. This requires the reinforcing to protrude over the support which should be concreted in the same operation. Saddle bars to provide continuity moment at the supports will also be required.

Where floors are to support truck loads, unless there is some physical barrier to prevent large trucks gaining access, consideration should be given to designing for HN loads. Floors constructed using 200 mm thick hollowcore units may not be appropriate for the shear forces associated with HN loads unless special measures are taken.

If all cores are broken out, temporary propping may be required for longer spans with thick topping.

Broken out cores and reinforcing to an extent exceeding that shown in NZS3101:Part 2:2006 figure C18.5 (Reinforced detail) may not be appropriate for use with ductile frame supports.

For spans in the 7 metre plus range where transport, access, location, lifting, building size, bay shape etc are suitable, Hollowcore flooring, with or without timber infills, will often provide the most economical solution.

TOPPING REINFORCING

The building designer is responsible for ensuring the performance of the completed building. This extends to the thickness and grade of concrete topping and the reinforcing in it, particularly where the floor is to act as a diaphragm to transfer horizontal forces to load resisting elements.

Although flooring suppliers may detail their recommended minimum topping reinforcing on their layout drawings or shop drawings, it should always be referred to the building designer for approval, and more than the minimum may be required where the floor is to act as a diaphragm to transfer lateral loads.

Reinforcing is required in precast floor toppings for the following reasons:

- Control of shrinkage
- Continuity reinforcing to minimise deflections
- Starter bars to tie in to the supports
- Diaphragms to transfer lateral earthquake and wind forces to load resisting elements

For low rise buildings, commonly used nominal topping reinforcing may be

- 665 HRC mesh throughout the topping
- starters from the perimeter HD 10 bars at 300 crs lapping 600 with the topping mesh
- saddle bars of HD 10 bars at 300 centres over all internal supports.

Greater levels of reinforcing should be used where appropriate.

Note NZS3101:2006 Commentary clause 18.6.7 requires the use of Grade 300 reinforcing for saddle bars for some seismic details to provide greater ductility and this may extend to starter bars.

During testing of a full scale floor, welded wire mesh was seen to “unzip” under simulated seismic deformation.

It is preferable to use grade 300 bars for topping reinforcing where the floor is to act as a seismic diaphragm rather than welded wire mesh which has limited ductility.

Where the supporting beams will be subject to rotation causing moment reversals at the supports, it is recommended that the same amount of reinforcing is continuous throughout the topping to avoid changes to negative moment capacity close to the supports. This means whatever saddle bars are over the supports are carried right across the floor and laps close to the supports are to be avoided.

CONTINUITY REINFORCING

The purpose of continuity reinforcing is to develop restraint moments at the supports and transfer diaphragm forces. It will have a major affect on deflections,

perceived “liveliness”

the interface between the underside of the precast unit and its support

compression or tension at the soffit of the floor at midspan and the supports

the shear capacity

flexural cracking occurring at midspan and the supports

the ability of hollowcore floor units to accommodate seismic induced support rotations

Development of restraint moments at the supports has a significant effect on floor performance and the end user’s satisfaction with the finished building. It is important that consideration is given to the detailing and reinforcing at the supports.

Restraint moments at the supports can only develop as the top reinforcing goes into tension. and where a corresponding compression force can be developed at soffit level.

For fully propped systems such as typical rib and infill floors or flat slab systems, restraint moments are developed by the self weight of the floor on removal of the temporary propping and then by application of applied loads.

In unpropped construction such as Tees and Hollowcore, the precast unit alone normally supports the self weight of the completed floor before any negative reinforcing can start to act. The negative reinforcing can only begin to activate and develop restraint moments on application of superimposed loads to the completed floor. With tees no appreciable restraint moments will be developed at any stage unless the webs are built in to their supports, and then the level of restraint moment developed may be governed by the torsional or flexural stiffness of the support. This significant difference to propped systems affects the level of restraint moments which can be developed, increases deflections, and may require deeper floors.

When considering the ability of a propped floor to develop a restraint moment, consider the moments developed by the full weight of the floor plus the superimposed loads. For an unpropped floor consider only the moment developed by the applied loads.

In unpropped systems such as hollowcore or double tees, the reinforcing in the topping can only be activated by superimposed loads occurring after the floor structure has been completed. A floor with self weight of 3.5 kPa may have a design live load of 1.5 kPa plus a superimposed dead load of 0.5 kPa. The average actual superimposed load may be only 1 kPa and the restraint moment developed and its affects will be relatively small.

Because floor systems are normally designed to meet the bending strength requirements from gravity loads in the simply supported condition, continuity reinforcing need not meet the requirements of the design code but should be designed so that under likely working loads, its yield strength will not be exceeded. In that case the floor will perform as fully continuous for deflections even though the continuity reinforcing does not meet full continuity moment requirements with load factors and capacity reduction factors applied.

Design of continuity reinforcing based on 1.0 G + 1.0 Q with capacity reduction factor of 0.9 or 1.0 should provide sufficient capacity to give deflection performance similar to a fully continuous floor.

As an approximation, using 35% – 40% of the factored dependable moment required at midspan for design of continuity reinforcing will provide acceptable performance. A more accurate assessment can be carried out if that gives an unreasonable level of reinforcing. In any case a minimum of 10 mm dia bars at 300 centres should be carried over the supports. This is the minimum recommended level to resist an accumulation of shrinkage forces at the support that can have adverse affects on the seating detail, as well as a minimum to maintain the diaphragm integrity.

For unpropped systems an assessment of the moment resulting from the superimposed loads only on the composite section should be carried out. This will frequently confirm that the recommended minimum of 10 mm dia bars at 300 centres is adequate.

In the occasional cases where either web mounted double tees or Hollowcore units are propped during placing of the topping, the reinforcing may be designed as for propped systems. Because the propping may not be intended to support the full weight of the floor during construction, it may not utilise the full amount of reinforcing.

In all cases the amount of topping reinforcing must be sufficient to meet the requirements for diaphragm action.

CAMBER

The residual camber in prestressed concrete sections is the small difference between the hog caused by the eccentricity of the prestress, and the sag caused by gravity acting on the mass of the unit. The small difference between two larger effects. Small percentage variations in those two components can produce relatively large percentage variations in the difference between them.

Prestress lift of 60 mm less gravity down of 40 mm results in a camber of 20 mm. If each component varies + or – 20%, the calculated camber of 20 mm can vary by + or – 20 mm, from 0 mm to 40 mm (+ or - 100%)

Each of these components is subject to a number of variables, some of which vary over time, are dependant on the load history, and are also subject to environmental effects.

There are the normal dimensional variations and tolerances. The effect of these on section properties may not be great, but the effect on eccentricity may be significant.

The modulus of elasticity of concrete is typically assumed to be a function of strength. It also appears to vary with age independently of strength. Concrete which has reached 25 MPa after 70 days will have a different elasticity at that time to 12 hour old concrete which has reached the same strength in a much shorter time.

The elastic properties are assumed to be a function of strength, which is variable, not just over time but is also affected by temperature, curing conditions and exposure.

Another major component is concrete creep, which can be either upwards or downwards. Concrete creep varies with the amount of stress applied, the age at which that stress is applied, and the duration. The main factors affecting the creep stress are self weight of the unit which can vary as moisture content varies, the amount of prestress force which varies over time with relaxation, drying shrinkage, accumulated creep, as well as applied loads. Concrete shrinkage also affects the stress, and the time at which this occurs and its duration. It is affected by environmental conditions as well as depending on the concrete materials.

Drying shrinkage of the concrete topping will cause the composite section to deflect downwards, but while this is occurring, the precast unit is normally in compression and creep shrinkage is occurring which causes the floor to deflect upwards. The net affect of these is difficult to predict with any degree of accuracy because of the number of variables involved that differ with each job and can differ each day.

Environmental effects are the exposure conditions the unit is subject to, particularly during the first months of its existence. The temperature variations will affect the strength gain. The drying conditions will affect the time at which shrinkage occurs. The top surface of the unit may be exposed to very different conditions to other parts of the unit and the properties affecting camber may vary throughout the depth of the unit. If units are stacked for any length of time, the top unit typically exhibits much greater camber than the units below it due to different exposure conditions even though they were all manufactured at the same time in the same bed with identical prestress and concrete.

Even when the finished unit is in service, temperature gradients caused by solar heat gain on the top surface where it is exposed to the sun can cause variations in camber. Daily fluctuations in excess of 30 mm have been measured in the level of a carpark deck from this effect, but it is not an issue in the majority of situations where floors are used in internal environments.

For prestressed units built into supports, the restraint effects from the supports will also have a significant effect on the camber.

Camber calculations are typically carried out using theoretical instantaneous assumptions, and factors based on experience applied to those figures.

Because of the variables involved, the resulting calculated cambers are subject to wide variations, and will vary significantly over time.

PENETRATIONS THROUGH THE TOPPING

The following notes are for general guidance only and the floor designer should be consulted regarding penetrations through the floor.

Penetrations may be formed by drilling through the completed floor, by forming penetrations in the precast unit prior to delivery and boxing out the topping, or by boxing out the topping and cutting the precast unit after the topping has been cast.

Wet cast products such as flat slabs and double tees can have penetrations cast in at the time of manufacture although in many cases, and particularly for smaller penetrations, it will be more economical to drill or cut them on site after delivery. This enables more accurate location and less concern about location of specific individual units.

Hollowcore units formed by extrusion or slide forming process cannot have penetrations cast in during the casting process. For these it is necessary to cut or drill all penetrations after the product has been manufactured. This may be prior to or after delivery to site.

The effect of penetrations must be considered when determining the strength of the completed floor.

Near the support, the shear capacity is more critical than the bending capacity, and towards midspan the bending capacity will be more critical.

Most floors can accommodate a reduction in compression flange at mid span without appreciable reduction in bending capacity.

Ribbed systems such as double tees and rib and timber infill should have penetrations located clear of the prestressed web or rib. Within the middle half of the span, particularly for highly stressed floors, penetration size may be restricted by the need for sufficient flange width to provide compression zone for the bending capacity. Close to the support large penetrations can be formed provided they do not compromise the prestressed web or rib, and the function of the continuity reinforcing. 2,400 wide double tees can typically accommodate 800 wide penetrations between the webs, 1,200 wide tees can accommodate 450 mm wide penetrations between the webs. Rib and infill systems with 900 centres can accommodate 650 wide penetrations, although greater widths can be used by local rearrangement of rib set out. Greater rib spacing requires longer infills that may require propping.

Flat Slab units Penetrations close to supports where the reduction in bending capacity is not critical may cut a prestressing tendon without compromising the strength as long as there is still sufficient slab for the shear requirements. Further from the supports where bending capacity becomes critical, penetrations should be formed between prestressing tendons. Tendons should not be cut within the middle half of the span unless additional capacity has been provided at the design and manufacturing stage and/or the manufacturer has given specific written approval. It may be difficult to determine the location of prestressing tendons after the topping has been cast. For this reason it is preferable for penetrations to be located and set out prior to casting the topping. Concrete cover to prestressing tendons is also a factor to be considered.

Hollowcore units Penetrations should be located to avoid cutting through the webs which is also where the prestressing tendons are located. At the support, the webs are providing the shear capacity although the unit may still have sufficient shear capacity after one web has been removed. Further from the support cutting through the web is also likely to cut through prestressing tendons and compromise the bending capacity. The units can be designed with additional prestressing tendons to allow for cutting of occasional tendons further into the span. Location of the webs is fixed and can be supplied by the manufacturer but after the topping has been cast they should be located from the underside of the slab. With care, penetrations of 120 mm diameter can be drilled through the cores without damaging the webs or the prestressing tendons. Hollowcore units have been manufactured to different profiles at different times, even by the same manufacturer so care needs to be exercised each time a penetration is to be cut through a hollowcore unit.

Spaced Hollowcore units Where Hollowcore units are spaced apart using timber or other infills, there is generally little restriction on penetrations through the infill.

SERVICES IN THE TOPPING

With the exception of heating systems, services are not normally cast into the in situ floor toppings and should not be used without the written approval of the manufacturer. In slab hot water heating systems are specifically designed for casting in and require additional topping thickness to maintain cover to reinforcing and integrity and additional reinforcing to cope with the increased thermal movements.

Casting services into topping concrete can compromise the diaphragm capability, the bending and shear capacity, the composite action, and lead to unacceptable cracking. Concrete cover to topping reinforcing is also an issue, particularly where different reinforcing layers overlap.

Sometimes water pipes are cast in using thicker topping and considering the issues above, but both hot and cold pipes should be lagged to accommodate movement and for insulation.

HEATING CAST INTO THE TOPPING

Only some aspects of this specialist subject are covered here. The heating system supplier should be consulted for further information.

Heated floor slabs provide a very comfortable form of background heating that can give years of trouble free service.

Heating within the slab will cause greater drying and thermal movements of the concrete topping. These in turn may result in increased cracking, but will not normally be of structural significance. All concrete shrinks and as it shrinks it cracks, but these cracks are not normally visible under most floor coverings but care should be taken with some, particularly vinyl.

Floors to be tiled should not have the tiles installed early in their life to permit normal shrinkage and the additional drying effects from the in slab heating to take place prior to installation.

Electrical heating elements may be cast into the topping or may be attached to the top surface of the finished floor under floor coverings or tiling. Where electrical heating elements such as Pyrotenax are cast into the topping, no additional precautions are required as long as appropriate topping thicknesses, diaphragm reinforcing and continuity reinforcing are used and adequate cover maintained.

Where heating elements are attached to the top surface of the floor no additional precautions are required. This system is sometimes used for localized heating such as trafficked zones under tiles in bathrooms. If this is the case the thermostat controlling the heating can measure the slab temperature rather than the air temperature. This has the effect of ensuring bare feet are walking on warm surfaces when air temperature thermostats may have switched off other heating.

In slab hot water heating systems distribute heat through hot water pipes embedded in the concrete topping. Because of the thickness of the water pipes, care is needed when locating them within the topping slab. The topping slab is part of the structural system providing the compression flange and additional thickness will be required to accommodate water pipes. Considering the topping reinforcing where layers of mesh must overlap, with continuity reinforcing and saddle bars, and cover requirements for protection of this reinforcing, at least 100 mm minimum topping thickness will be required to accommodate hot water in slab heating.

Even greater thickness may be required if water pipes cross over each other within the topping.

In slab hot water heating may also be used for cooling in the summer if a reverse cycle system is used.

Concrete floors provide a heat sink to even out temperature fluctuations. In normal applications the underside of the floor is within a building and insulation may not be required.

THERMAL INSULATION

Concrete floors are a good heat sink and have the advantage of storing energy during periods of warmth and releasing it when temperatures drop. This “thermal inertia” evens out daily temperature fluctuations to increase comfort levels.

The insulation values for precast concrete floor systems is similar to that obtained from solid concrete, and further steps need to be taken if more insulation is required. It is not usually required between floor levels within a building, as heat loss between enclosed floors is not normally an issue. It may be required where the floor acts as an exposed roof or the underside is exposed to the exterior such as over open car park areas.

With rib and infill floors, the timber infill provides some insulation and additional insulation can be obtained by using a layer of polystyrene (typically 20 mm to 50 mm) on top of the infills. It is necessary to glue it down to prevent it moving or floating during placing of the concrete topping. If this is to be used, the floor supplier should be notified prior to manufacture to enable higher stirrups to be installed in the ribs. Insulation should not protrude on top of the prestressed rib as it will impair bond between the rib and the in situ topping.

For other floor systems, if additional insulation is required, it must be provided within the ceiling, or by attaching insulation to the underside or top of the completed floor.

ACOUSTIC

At the time of writing the Building Code Clause G6 – Airborne and Impact Sound was being drafted and should be consulted for all acoustic design matters as some of the information in the following paragraphs may be superseded.

Precast concrete floor systems provide good acoustic barriers.

Previously, there were two acoustic considerations. The Sound Transmission Coefficient (STC) rating which measures the resistance to airborne sound, and the Impact Insulation Class (IIC) rating which considers the effect of an impact on the surface.

Flat slabs, hollowcore units and some spaced hollowcore systems provide better than 55 dB STC rating with minimum topping thicknesses of 65 mm over the precast unit. Some hollowcore profiles have been modified to allow increased thickness locally over the infills to retain the acoustic and fire resistance ratings.

Double tees with 50 mm flange thickness typically require 80 mm of topping to achieve 55 dB STC rating, and rib and timber infill systems with 75 topping require a nominal 9.5 mm gib board ceiling for the same rating.

In common with all floor systems, not just precast concrete, the floor can only be isolated from direct impact by the use of floor coverings. IIC ratings for inter tenancy floors depend on the floor coverings used.

TOPPING THICKNESS

Minimum topping thickness is governed by requirements of cover to the topping reinforcing, structural considerations for diaphragm action, and in some cases acoustic or fire rating issues.

Reinforcing cover requirements dictate 75 mm minimum thickness for rib and infill systems, and 65 mm for others. Greater topping thickness is required over timber infills to provide concrete cover below the reinforcing whereas the precast unit provides bottom concrete cover in other systems.

Where mesh reinforcing is used, there will be at least three layers at overlap locations, and when starters from supports and saddle bars are considered, these thicknesses allow little room for tolerance to maintain adequate cover to all the reinforcing.

For exterior use, particularly where the top surface is exposed without a waterproof membrane, increased topping thickness should be used. Refer to NZS3101:2006 Section 3.

These minimum topping thicknesses will provide 60 minutes fire rating for 75 mm flat slabs and 90 minutes or greater fire rating to most other precast concrete floors.

Where 55 dB STC rating is required additional topping thickness or other measures will be required for double tees or rib and infill floors.

For units such as double tees or hollowcore, topping thickness may be reduced slightly at midspan to accommodate camber or camber variations provided care is taken to ensure cover and other requirements are not compromised. This may be a matter of ensuring reinforcing laps are minimised at these locations.

TOPPING LEVELS

Propped Systems. Removal of propping from a floor system after the topping has been cast will cause the floor to deflect. See also “CALCULATION OF PROPPING EFFECTS”. For propped floor systems such as rib and infill are flat slabs, the concrete topping is normally cast to follow the camber of the propped precast unit so that it is not immediately dished on removal of the props.

The purpose of propping the floors to a camber, is so that on removal of the props after the topping is cast, the top surface which drops the same distance as the floor deflects, is closer to a level surface.

Imagine the floor is propped level and the topping is also cast level. On removal of the props, the floor would deflect, and over time with application of loads together with creep and shrinkage it will deflect further. Ideally the floor would be precambered to compensate for that deflected shape but that is often complicated and difficult to predict with any degree of accuracy due to the effects of different supports at right angles and parallel to the span as well as restraint effects. Shrinkage, creep, timing and duration of loading all have an impact which make attempts to construct to a camber to compensate for depropping and other deflections an approximation at best. The ability to form finished toppings to accurately curved shapes to compensate for deflections is limited and expectations of the levelness of finished floors should be realistic.

The levelness of the finished floor is also affected by deflections of the supporting beams, which may also be subject to depropping deflections.

Manufacturers provide suggested precambers for their floors, but these are only approximations which become more so as spans increase or slender floors are used. If better accuracy is required, discuss with your suppler and consider increasing the floor depth.

Unpropped Systems. The topping on unpropped systems such as Tees and Hollowcore is normally cast level. Once the floor is loaded, it will deflect. These deflections are generally much smaller than with propped systems and the difficulties of attempting to cast the topping shape to compensate for the smaller deflections outweighs the likely benefits.

Casting the topping level over units that have a natural camber results in additional topping thickness at the supports. The average thickness of the additional topping is approximately 1/3 of the maximum camber. If the unit is cambered 30 mm, casting the topping level with the required thickness at mid span would result in an average of 10 mm of additional concrete. In some cases the minimum topping thickness is reduced by 10 mm at mid span thereby compensating for 30 mm additional concrete thickness at the supports due to the camber of the unit. Reduction of the topping thickness locally should not be done without reference to the designer.

Temporary propping levels and support levels of components will need to be adjusted to compensate for cambers of the floors and beams. The levels of tops of columns and walls may need to be lowered where beams and floors are seated onto them.

FIRE RATING

Fire ratings can be obtained by compliance with NZS3101:Part 1: 2006 Section 4 “Design for Fire Resistance”, or by testing. Some products have been tested by the manufacturer and achieved better fire ratings.

75 mm flat slabs with typical topping provide a 60 minute fire rating. Most other precast floor systems with typical topping provide at least a 90 minute fire rating. Higher fire ratings are available and should be referred to the local manufacturer.

DURABILITY

Manufacturers Load / Span tables for floors are based on the concrete cover required for internal environment exposure classifications as that is the typical environment for floors. Where different exposure conditions apply, the increased cover requirements may alter the maximum load carrying capacity.

Most precast floor systems can be manufactured to meet the requirements of NZS3101:2006 Section 3 for 100 year life in internal environments. Refer to your local manufacturer for specific advice.

Most precast floor systems will meet the requirements for 50 year life for B1 (Coastal perimeter) environment, and products meeting the requirements for 100 year life can be provided. Refer to your local manufacturer.

Factory made prestressed products will generally provide greater durability compared to some other concrete construction because of the improved quality of factory manufacture (better tolerances, controlled manufacturing etc) and will normally remain uncracked.

For exterior exposures, designers should consider cover requirements of reinforcing in the in situ topping. If suitable waterproof membranes are used this may not be an issue.

SAW CUTS IN TOPPING

Saw cutting of the topping concrete of suspended slabs is not normally recommended.

Slabs on the ground are normally cut to control shrinkage cracks. They do not prevent cracks. Their objective is to initiate the crack at the saw cut so that it is below the saw cut and not seen as an unsightly random occurrence. They are used purely for visual effect.

Topping slabs to suspended floors are subject to flexural as well as shrinkage cracks. The number of likely shrinkage cracks is greater within the thin and lightly reinforced in situ slab so more saw cuts would be required to be effective, and their location will be influenced by the pattern of precast elements beneath them. Additionally there will be flexural cracks close to supports, and diagonally across the corners of bays from the holding down effect at the corners.

The greater number of likely shrinkage cracks, combined with the difficulty of accurately predicting precise location of the shrinkage and flexural cracks means that attempts to hide cracks in suspended slab toppings will require a large quantity of saw cuts, and are likely to be only partially effective.

For saw cuts to be effective in initiating cracks, they must penetrate part way through the thickness of the topping slab, and in the process they are likely to cut topping reinforcing and reduce the concrete cover allowing earlier initiation of corrosion. They may also compromise the effectiveness of the diaphragm action.

Saw cuts in the topping slab may also compromise the compression flange.

The cost of saw cuts is unlikely to provide commensurate benefits, and are likely to cause a reduction in floor performance and durability.

PRETENSION PRESTRESS PROCESS

Pre tensioned precast floor systems are manufactured in long runs. Hollowcore beds may be 100 to 200 metres long. High strength 7 wire prestressing tendons are anchored at each end and stressed. In a 100 metre bed they may be stretched 700 mm during the stressing process and 12.9 mm dia tendons may have 130 kN force applied.

The concrete is cast after the tendons are stressed, and then heated to achieve sufficient strength overnight to enable it to provide sufficient bond to prevent movement of the tendons when they are released. As the tendons are released the concrete product is pulled along the bed by the elastic shortening of the tendons, This movement requires highly accurate moulds to prevent the hardened concrete jamming while sliding. Because the finished product slides along the mould, it limits the ability to cast in items or cope with irregular shapes.

Prestressed tendons are normally released gradually. If they are simply cut while under high stress, the shock from this sudden release can cause slippage, splitting or other damage to the concrete in the bonding zone.

The distance they retract along the bed when the tension is released depends on the length of tendon that is cast within the concrete product and the length which is free. As the stress is transferred to the concrete, the concrete will shorten, but by a much lesser amount than the free tendon.

As tendons are stretched, they become slightly thinner, and as they are released they expand again. This expansion, combined with the shape of the 7 wire tendons, results in development of their full stress over a relatively short distance within the concrete product.

At the end face of the concrete product, the stress in the tendon is zero, but it rapidly develops to a high level within a short distance. As the tendon pulls in to the concrete, it exerts splitting forces that can damage the ends and cause loss of prestress.

The concrete must have sufficient strength at the time the prestress is released to resist these splitting forces without supplementary or confining reinforcing. If it has sufficient strength for this, and for the handling processes during removal from the mould and transport to storage, it should have sufficient strength for its final use. For this reason under strength concrete will normally result in obvious failure before the unit is stacked in the yard and is unlikely to ever be an issue with units delivered to site. Thus a level of quality control is automatically assured in prestressed concrete.

Reinforced (as distinct from prestressed) concrete cracks when it is subject to tension above a certain level and deflections will increase under normal loading conditions due to the very different section properties of the cracked concrete sections.

Pretensioned high strength concrete will normally remain in compression during use, resulting in improved deflection characteristics and as a consequence thinner lighter sections can be used.

Combining high strength concrete, with high strength prestressing tendons gives superior structural performance. When this is added to quality controlled factory production and multiple mould re use, it results in a very cost effective outcome.

Diagonal tension resulting from shear is better resisted by precompressed concrete providing savings in shear reinforcing.