Seismic Performance of Hollow-core Flooring: the Significance of Negative Bending Moments

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ABSTRACT: Hollow-core flooring units, as described in the technical literature, are intended to be used as simply supported members. However, in construction continuity is often established between the units and supporting structure by the addition of insitu topping concrete and reinforcement. This change in structural form can result in negative moments and axial forces being induced in the floor by gravity loads, wind and seismic actions. Vertical seismic ground motion in particular can make a significant contribution to negative moments induced in the floor. This paper focuses on two failure mechanisms which may occur in negative moment regions of hollow-core floors, namely a flexural failure and a shear failure. It is shown that, with the detailing in common use prior to the release of the Structural Concrete Standard, NZS 3101-2006, there is a potential for brittle negative moment failure to occur under seismic conditions. Analytical work indicates that under some conditions a diagonal tension (shear) failure may also occur. As the failure of a floor may lead to progressive collapse it is important that these two aspects are considered along with a number of other potential failure modes in the retrofit or design of buildings. Guidance is given on methods of assessing the negative moment flexural strength and shear strength of hollow-core floors.

1 INTRODUCTION

Hollow-core units are frequently used to form floors in multi-storey buildings as they are economic, have good sound and thermal properties and long spans can be achieved economically. The poor performance of buildings with hollow-core floors, backed up by the premature failure of a floor in a structural test, has demonstrated the vulnerability of these buildings and highlighted the need for research into this area (Iverson and Hawkins 2004, and Matthews 2004). There are a number of issues, which need to be addressed in the design of new structures or retrofit of existing construction containing hollow-core floors. These are summarised in a Department of Building and Housing (DBH) Report, which is due to be published early in 2008. There is a need for a deeper understanding of how hollow-core floors perform in earthquakes so that a hierarchy of strength can be assessed with confidence. With this knowledge a capacity design approach can be used to protect the floors from a suite of potential failure modes in a major earthquake. Research on hollow-core floors performed in New Zealand over the last 15 years has increased awareness of their potential vulnerability and this has led to revisions in the way hollow-core flooring is designed and installed. However, much of this is empirical and the actual mechanics of hollow-core floor behaviour is not fully understood.

In this paper two, of a suite of potential failure modes, are considered; These failure modes, namely a flexural failure and a shear failure can only occur in negative moment zones. This sensitivity occurs as the vast majority of hollow-core units used in New Zealand have been reinforced with pretensioned strands placed close to the soffit. Top strands have been omitted. As a result flexural cracking can occur at low negative moment levels. Where non-ductile reinforcement, such as mesh, is used in the topping concrete, a brittle flexural failure can occur. In other situations negative moment flexural cracking results in the shear strength of the member being limited by flexure-shear cracking rather than web-shear cracking. Web-shear cracking is normally the critical condition of shear strength in positive moment regions close to the support of simply supported members. This change is critical as the flexure-shear cracking strength is much lower than the web-shear cracking strength. A consequence of this is that shear strength tests of simply supported hollow-core units are not applicable to the situation where continuity is established at the supports.

The conclusions drawn in this research are based on experimental work and analytical investigations into the performance of individual hollow-core units with reinforced concrete topping.

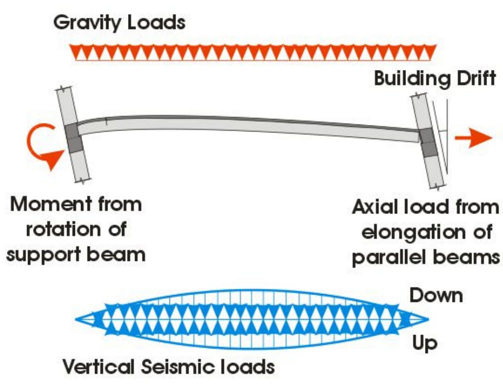
2 NEGATIVE BENDING MOMENTS IN HOLLOW-CORE FLOORS

Negative moments can be induced in hollow-core floors, when continuity has been established by the addition of reinforced topping concrete, by a number of different mechanisms. The most important actions are;

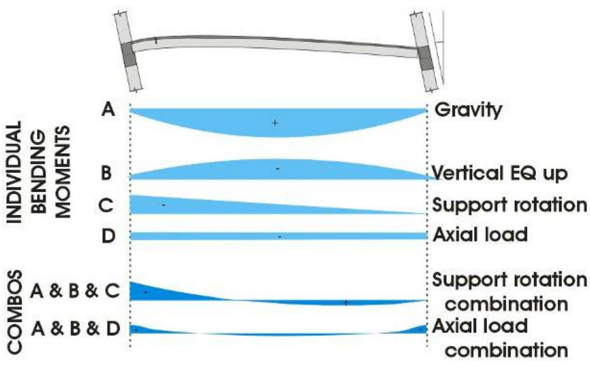
- Sway of the building, due to wind or seismic actions, which induces relative rotation between the support and hollow-core units;
- Vertical seismic ground motion, which induces upward and downward seismic forces on the floor;
- Elongation of beams parallel to the hollow-core units, which pushes the supporting beams apart. This induces tension in the starter bars connecting the topping concrete and hollow-core units to the supports. This action induces axial tension and negative moments in the floor.

Other actions, which may induce negative moments in hollow-core floors, but are of lesser importance in the ultimate limit state, include creep redistribution due to the change in structural from (converting the units from simply supported to members with continuity at supports), and actions arising from differential shrinkage of precast and insitu concretes and differential temperature conditions. The significance of this group of actions generally becomes negligible when advanced flexural cracking has developed at the support points. This greatly reduces the flexural stiffness, which reduces negative moments associated with the second group of actions.

Figure 1a shows the potential forces on a hollow-core floor due to the first group of actions. To find the critical negative bending moments and axial loads in a hollow-core floor system, the bending moment components from the individual loads are superimposed. Figure 1b shows the individual bending moments and their critical combinations. Two combinations are shown, one including axial tension (induced by parallel beam elongation), labelled A&B&D, and one including a moment induced at the support by support beam rotation, labelled A&B&C.



(a) Loads that contribute to negative bending moments



(b) Bending moments from potential loads acting on a hollow-core floor and their combinations

Figure 1 Possible negative bending moments induced in hollow-core floors

The seismic design actions due to vertical ground motion can be found using the New Zealand

Structural Design Earthquake Actions Standard (Standards New Zealand. 2004b). The values to use for the structural ductility, μ, and the structural performance, S_p, factors vary depending on the steel reinforcement used in the insitu topping. The authors recommend that a structural ductility factor of 1 is used where the reinforcement in the topping is mesh and 2 where ductile grade E reinforcement is used. The associated S_p factors should be determined from the structural ductility factors. When representing dynamic actions as equivalent static forces, the magnitude of each force is proportional to the product of the mass and deflection relative to the ground. For a hollow-core floor, where the mass is approximately uniformly distributed, the deflected shape is approximately a parabola. Hence, for practical purposes, the vertical seismic actions may be distributed in the shape of a parabola along the length of the floor (Fenwick et al. 2004). The bending moments resulting from upward vertical seismic actions are shown as "B" in Figure 1b.

The actions transmitted to a floor through the interface between the hollow-core unit and the supporting beam in a major earthquake are likely to cause the reinforcement in the insitu topping to reach a stress close to its ultimate value. The moments and axial force acting in the floor will vary continuously during an earthquake and depend on the connection detail used. However, two critical situations are considered appropriate for design or assessment, these are:

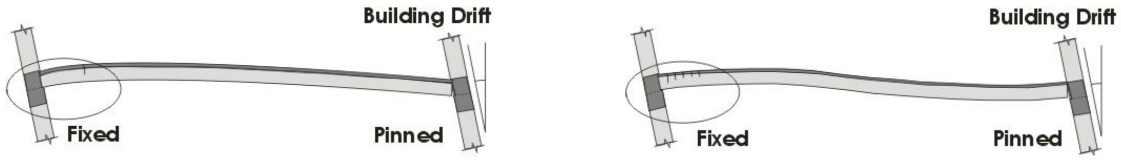
- Maximum bending moment with no axial load: This scenario could be induced by rotation of the supporting beams due to building drift. In this case it can be assumed that one end of the floor is at its overstrength moment and the other is pinned (zero moment), with a linear variation in between. This is shown as "C" in Figure 1b. The overstrength bending moment capacity at the floor end can be assessed assuming that the interface section acts as a singly reinforced concrete beam section.
- Maximum axial load due to the elongation of beams parallel to the precast floor units: In this scenario end moments in the floor are induced by the eccentricity of the applied axial tension through the starter bars. The shape of the bending moment caused by the axial load and eccentricity is shown as "D" in Figure 1b.

It is conservative to assume overstrength end moments due to elongation or rotation of the support beams occur simultaneously with the maximum vertical earthquake excitation. This is realistic assumption as the fundamental period of the floor, excited by the vertical motion, is short compared to the fundamental period of the structure due to lateral seismic excitation. It is therefore likely that both maximum moments occur at the same time. During the downward vertical seismic actions high shear stresses can be induced near the supports in negative moment zones.

3 FAILURE MODES OF HOLLOW-CORE FLOORS POSSIBLE WHEN SUBJECTED TO NEGATIVE BENDING MOMENTS

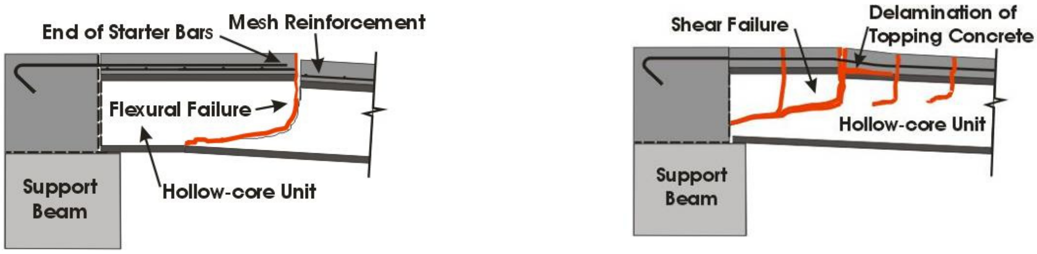
The two failure mechanisms are illustrated in Figure 2. The critical section for a negative moment failure is at the end of the starter bars. The magnitude of the moment that can be induced at this position depends on the point where the starter bars are terminated, the magnitude of the tension force that may be resisted by the starter bars at the support and the magnitude of the vertical seismic forces acting on the floor. As described in the following sections, the negative moment strength can not be determined by conventional flexural theory where mesh is used in the topping concrete, due to its limiting strain capacity and tension stiffening of the concrete. When a brittle flexural failure occurs the crack runs down to mid depth and then it follows a diagonal path as the unit tries to sustain the shear force in the compression zone. This diagonal crack results in collapse. Hence, this form of failure **cannot** be neglected on the erroneous basis that when the moment reverses the crack will close enabling positive moment and shear to be resisted.

A flexural shear failure may occur as illustrated in Figure 2 (b) and (d). In this case a series of flexural cracks are induced in the concrete. The tension force in the reinforcement in the topping concrete is a maximum above the support. The decrease in this tension force away from a support applies shear to the concrete in the tension zone. This can lead to flexure-shear cracking and failure. Assessment may be made using the design rules for prestressed concrete with the added complication that this occurs within the development length of the pretensioned strands. However, as indicated in a later section such an analysis would lead to a conservative assessment of shear strength.



(a) Hollow-core floor subjected to earthquake drift under upward vertical earthquake motion

(b) Hollow-core floor subjected to earthquake drift under downward vertical earthquake motion



(c) Flexural failure when mesh reinforcement ruptures at the end of starter bars

(d) Flexure-shear failure induced by change in tension force in steel reinforcement between flexural cracks

Figure 2 Potential failure modes of hollow-core flooring when subjected to negative bending moments

4 EXPERIMENTAL INVESTIGATION OUTLINE

To investigate the two failure modes described in Section 3 a sub-assembly unit representing a segment of hollow-core floor was tested for each mode. The test units comprised of a 6 m long length of 300 mm deep hollow-core unit with insitu concrete topping, supported at one end by a length of seating beam. The support beam was fixed to the laboratory strong floor and the hollow-core test unit was loaded by three hydraulic actuators, as illustrated in Figure 3. The vertical actuators allowed different bending moment profiles to be applied to the specimens, while the horizontal actuator allowed the application of axial tension.

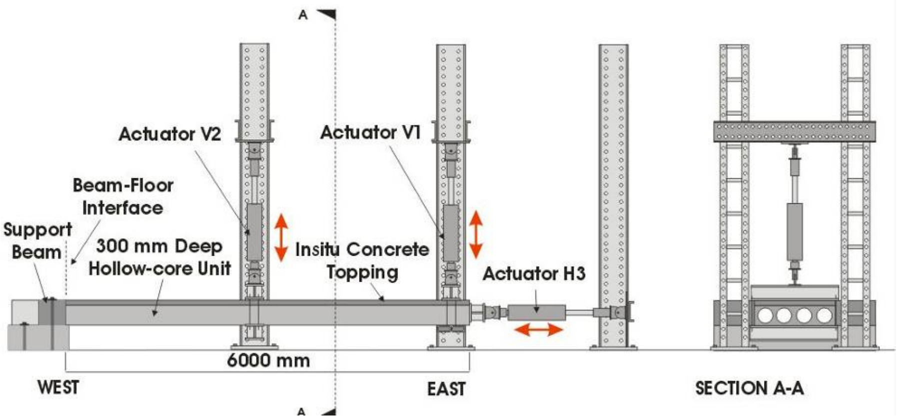
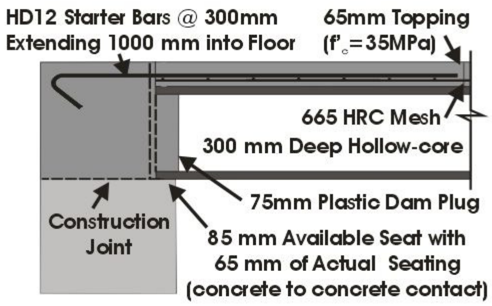
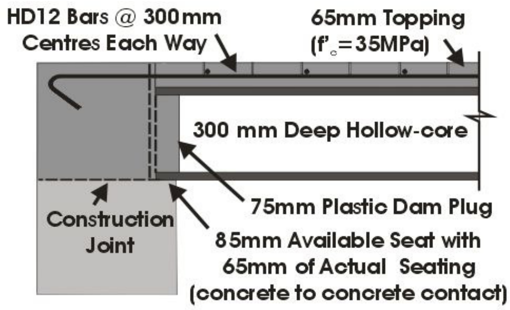


Figure 3 Single hollow-core unit and seating beam sub-assembly test rig used to subject the test unit to different load combinations

The same test rig was used for both experiments. However, the connection details between the support beam and unit, and the loading protocols were varied. The first test was performed to investigate the possibility of a negative flexural failure and it is referred to as test HCW1. The connection detail used for this test is illustrated in Figure 4a. This detail was designed to represent hollow-core floor connections common in New Zealand in buildings constructed prior to the 2004 amendment to the Concrete Structures Standard (Standards New Zealand 2004). A crack initiator was placed in the insitu topping concrete at the end of the starter bars during casting of the concrete to allow instrumentation to be positioned at the crack. The second test, which is referred to as test HCW2, was completed to examine a flexure-shear failure in a negative moment zone. The connection detail used for this test is illustrated in Figure 4b. Crack initiators, which extended from the surface of the insitu concrete to the top of the steel reinforcement, were placed at 150 mm centres in the first 1.5 m from the support point. These initiated cracks were not expected to have an adverse effect on shear strength results, as had the specimens been older cracks would have been induced due to creep and shrinkage of the insitu concrete.



(a) Test HCW1



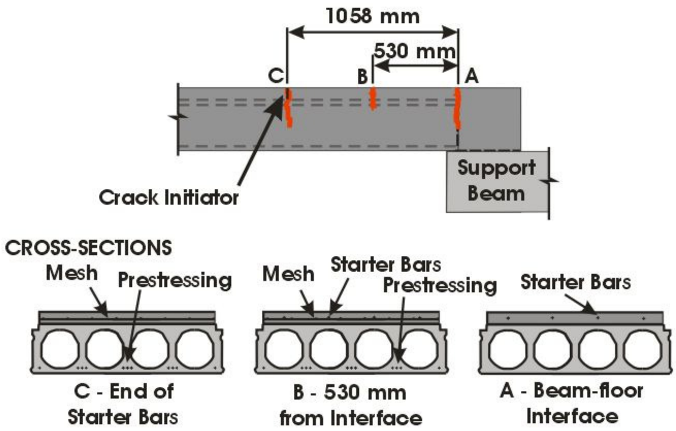
(b) Test HCW2

Figure 4 Connection details used in the two experimental tests

The loading history applied to the test of unit HCW1 was force based and it was completed in two stages. The first stage involved linearly increasing the negative moment at the support, while in the second stage negative moments were increased while the member was subjected to axial tension. The loading history for unit HCW2 was also force based, though in this case cyclic loading was applied to represent varying intensities of vertical seismic forces. The intensity of the vertical forces was varied to correspond to the design level actions corresponding to a medium seismic zone (Christchurch) and a high seismic zone (Wellington). The cyclic loading was intended to create a series of flexural cracks in the negative moment zone. However, very few cracks actually formed and it was not possible to generate a shear failure in the unit.

5 NEGATIVE FLEXURAL FAILURE

The test unit HCW1 contained 665 mesh reinforcement in the insitu topping concrete. This reinforcement had an ultimate strain capacity of the order of 2%. Grade 500E, 12 mm reinforcing starter bars extended 1000 mm into the topping concrete, connecting the support beam to the hollow-core floor. Under a negative bending moment, the test unit formed flexural cracks in three locations, as shown in Figure 5a. These were at the beam to floor interface, 530 mm out from the interface and at the end of the starter bars and they formed in that order. Only mesh reinforcement crossed the crack at the end of the starter bars and it was observed to yield immediately when this crack formed. This suggests that the tensile capacity of the concrete was more than that of the steel crossing this section. A consequence of this was that yielding of the mesh was limited to this one location, which limited the ductility of the test unit. When increasing axial tension and negative bending moments were applied to the test unit, the crack at the end of the starter bars continued to widen until the mesh ruptured and the specimen failed in a brittle manner. Figure 5b and c show photos of the failure once some of the insitu concrete down the sides of the unit had been removed. This failure occurred at an axial load and bending moment combination that were less than the calculated actions corresponding to design level seismic actions for Wellington for the ultimate limit state (return period 500 years).



(a) Locations cracks formed in test specimen HCW1 during testing and cross-sections of the specimen at these locations

(b) North side after failure

(c) South side after failure

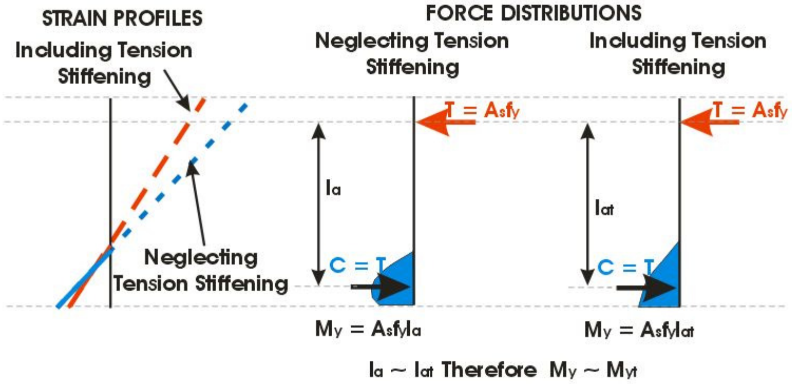
Figure 5 Test specimen HCW1

Of particular significance was the observation that the experimentally measured flexural strength of the unit was only 50 to 75 % of the strength derived from standard flexural theory. The two limits are given as there was some doubt on the magnitude of bending moment due to not knowing the exact weight of the test unit. It should be noted that in the test, a strength reduction factor of unity was used and the actual ultimate strength of the mesh (630 MPa). In design, flexural strengths would be based on lower characteristic strengths (485 MPa for mesh) with a strength reduction factor of 0.85, which would further increase the discrepancy between the predicted and measured flexural strength. With standard flexural theory it is assumed that plane sections remain plane and stresses are uniquely related to strain. In this project the stress strain characteristics of the mesh were modelled by a bi-linear relationship, with a yield stress of 570 MPa and an ultimate at 2 % strain of 630 MPa. The stress strain relationship for the concrete was based on the Mander model (Mander 1982). The measured strength of the concrete in the precast unit was 88 MPa.

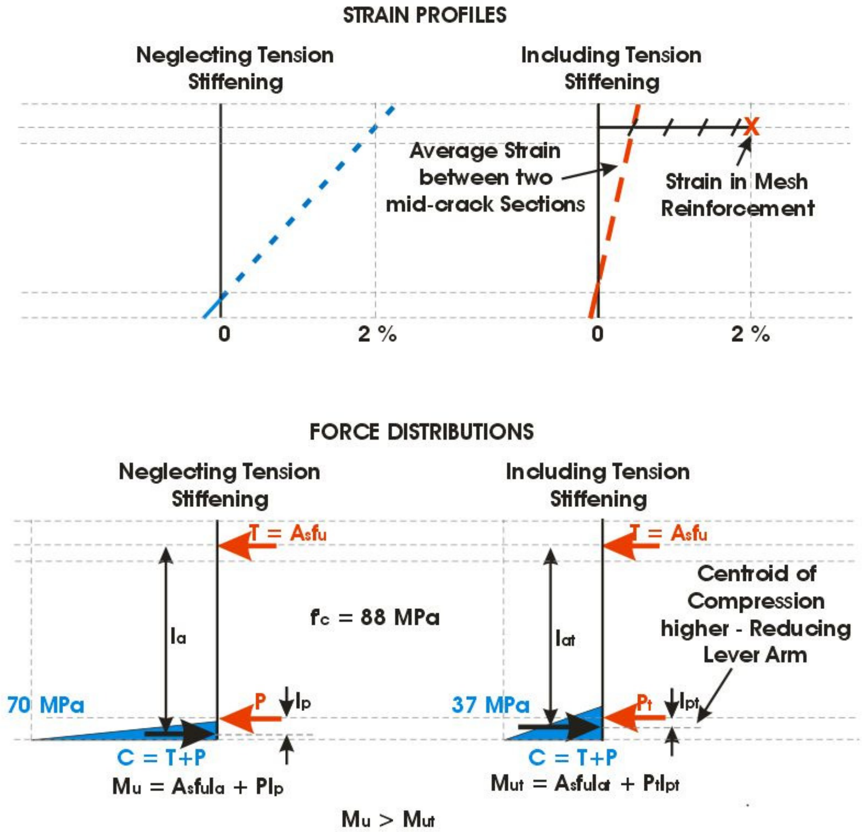
Figure 6a illustrates the situation in a typical reinforced concrete beam. Tension stiffening reduces the average strain in the tension zone and as such it has a minor influence on the location of the neutral axis and the stress distribution in the compression zone. However as shown in Figure 6a, the change in internal lever arm is small when tension stiffening is included in the analysis and there is no difference in the magnitudes of the tension and compression forces. Consequently tension stiffening can be ignored in strength calculations for practical purposes for typical reinforced concrete beams as it makes no significant difference to the strength.

This conclusion does not hold in the assessment of the negative moment flexural strength of hollow-core floors with topping concrete containing mesh. Using standard flexural theory and ignoring tension stiffening the ultimate strength corresponds to the situation where the strain in the mesh reaches 2 %. At this condition the predicted stress in the extreme compression fibre was 70 MPa. The centre of the compression force, which balances the tension forces in the mesh and pretension strands, is 10.25 mm above the bottom fibre. The bending moment resisted by the mesh at its ultimate stress level is 33 kNm and by the prestress force 35 kNm giving a total of 68 kNm.

To allow for tension stiffening it is necessary to consider how this influences the position of the neutral axis. This can be assessed by looking at how the average strain is reduced by this stiffening. The observed crack spacing was close to 500 mm. This value is consistent with the expected crack spacing for primary flexural cracks. The low tensile strength of the mesh was insufficient to generate secondary cracks in the concrete. The maximum tension force that can be sustained by the mesh (176 mm^2 at 630 MPa) could only induce an average tensile stress in the insitu concrete of 1.4 MPa, which is well below the anticipated direct tensile strength of the concrete (3 MPa). Hence, assuming the ultimate strain in the mesh restricted to a 150mm length (the distance between where the bars are welded to transverse reinforcement), the ratio of peak strain to average strain, known as a strain concentration factor, is of the order of 500/150, or 3.3. However, making allowance for bond stresses in the 150 mm length increases the predicted ratio of ultimate strain to average strain to 4.2.



(a) Strains and forces in a typical reinforced concrete beam



(b) Strains and forces in a prestressed hollow-core floor with topping containing mesh reinforcement

Figure 6 Effect of tension stiffening on members in bending

Figure 6b illustrates the case where the influence of tension stiffening is included in the strength calculation by using a strain concentration factor of 4.2. In this case the height of the neutral axis is increased and the stress in the concrete at the extreme fibre is 37 MPa. The height of the compression force increases to 17.5 mm above the soffit. With these strains the moment resisted by the mesh is 34 kNm and by the pretension force 21 kNm, giving a total of 55 kNm which is 80 % of the theoretical value ignoring tension stiffening.

To account for the effect tension stiffening has on the negative flexural capacity of a hollow-core floor, where the insitu concrete is reinforced with mesh, it is suggested that the negative moment flexural capacity be calculated using a strain concentration factor of 4. Further research may enable this value to be refined. Where Grade E deformed reinforcement is used higher strains may be sustained at the ultimate limit state and hence no strain concentration factor is required.

6 FLEXURE-SHEAR FAILURE IN A NEGATIVE MOMENT ZONE

In the HCW2 test unit, continuous Grade 500E, deformed 12 mm bars were used in the insitu topping. The loading sequence, in which cyclic loading was applied, was designed to induce negative moment flexural cracks in the region near the support and then subject this zone to high shear forces. However, it was not possible to induce significant flexural cracking in this zone and this prevented the formation of a flexural shear crack.

An analytical investigation was undertaken to predict the magnitude of the shear stresses sustained in the flexural tension zone of the beam. Separate analyses were made for 300 mm hollow-core units with 75 mm of topping concrete containing 2, 4, 6 and 8 Grade 500 bars with a diameter of 12 mm bars. These analyses were for the load case where the reinforcement in the topping at the support was sustaining its ultimate stress and the shear force in the negative moment region corresponded to gravity load and vertical seismic forces. In all cases, the predicted shear stresses in the webs of the hollow-core units were considerably in excess of the design levels for nominal shear strength given in the New Zealand Concrete Structures Standard, NZS 3101-2006.

It has been demonstrated that the shear strength of prestressed and reinforced concrete beams without web reinforcement depends on the width of the cracks in the tension zone (Collins 1999). This dependence arises as the shear stress that can be sustained by aggregate interlock action across the cracks increases as the crack width decreases. On this basis it is predicted that the negative moment region shear strength of hollow-core units with insitu concrete topping is greater than the corresponding values for reinforced concrete beams for the following reasons.

- Provided the topping reinforcement does not yield in the critical region the tension stiffening of the concrete significantly reduces the average strain in the reinforcement and this in turn reduces the widths of the flexural cracks.
- The highest shear stresses in the flexural tension zone are induced in the thinnest portion of the web. However, this portion of the web is located close to the zero strain line in the unit and hence the crack widths in this zone are small and consequently relatively high shear stresses can be sustained by aggregate interlock.

A tentative suggestion is that the nominal shear stress in the region of hollow-core floors subjected to negative moments be limited to $0.2\sqrt{f_c'}$. This is provided that the reinforcement in the negative moment region, excluding the portion between the edge of the support and a distance of an effective depth from the edge of this support, cannot yield under any seismic load condition. If yielding occurs in this region, in any loading situation, the crack widths are increased and the shear strength reduced.

7 CONCLUSIONS

1. Hollow-core units are designed to act as simply supported members. However, generally in construction continuity is established between the units and the supporting structure. This enables actions, which the units have not been designed to resist, to be induced in them.
2. Sway of a structure due to earthquake actions together with inertial forces associated with vertical ground motion can induce negative moments in hollow-core floors. Analyses show that the moments induced by these actions can in many cases exceed the negative moment flexural strength of the floor in the ultimate limit state.
3. A test of a hollow-core unit with insitu concrete topping, which was reinforced with mesh, failed in a brittle manner when subjected to negative moments. It is shown that the negative moment flexural strength cannot be predicted by conventional ultimate strength flexural theory. To predict the strength it is necessary to allow for the limited strain capacity of the

mesh and to allow for the adverse influence of tension stiffening on the strains in the mesh.

4 A test of a hollow-core unit with insitu concrete topping reinforced with ductile Grade 500 reinforcement indicated that the shear strength in the negative moment region was greater than would be anticipated from values for prestressed concrete beams based on the flexure-shear cracking strength. This increase in shear strength can be tentatively explained in terms of the crack widths in the critical regions of the web. It is tentatively recommended that the nominal shear strength of negative moment zones be limited to $0.2\sqrt{f_c^{'}}$ provided the reinforcement in the critical section (d out from the face of the support) cannot be subjected to yield under any seismic load combination. Further research on this aspect is required.

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