Alexandria Engineering Journal (2018) 57, 309–320

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Alexandria Engineering Journal

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ORIGINAL ARTICLE

Flexural behavior of precast concrete sandwich panels under different loading conditions such as punching and bending

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Received 25 May 2016; revised 22 September 2016; accepted 6 November 2016
Available online 5 December 2016

KEYWORDS

Punching;
Bending;
Precast;
Concrete sandwich panel;
Insulated structural panel;
EPS

Abstract Precast concrete sandwich panels having two wythes separated by a core may serve dual purposes of transferring load and insulating. Research studies with respect to flexural behavior of these panels under four-point bending are available in the literature. Nevertheless, experimental and analytical studies with respect to flexural behavior of concrete sandwich panels under punching load are not found. In this paper experimental and analytical studies carried out to understand and compare flexural behavior of concrete sandwich panels under two different loading conditions such as punching and four-point bending are presented and discussed. Experimental study indicates that, type of loading conditions affects the flexural behavior of the concrete sandwich panels significantly. The panel subjected to punching load failed in flexural mode, and its behavior is similar to conventional RC slab. Under four-point bending the panel failure is attributed to failure of concrete by combined effect of shear and flexural stresses. For both types of loading conditions, analytically predicted cracking moment is comparable to the experimental cracking moment. Further experimental and analytical studies are required in this area to develop design guidelines for practical applications of these types of panels under different loading conditions.

1. Introduction

Precast concrete sandwich panels, also known as Insulated structural panels, consist of two skins of concrete called as wythe separated by a core made of Expanded PolyStyrene (EPS) that provides significant thermal and sound insulation. Wythes may be reinforced by using welded wire mesh or conventional steel rebars. Composite action of the panels may be achieved by connecting the wythes using discrete or continuous shear connectors [1] made of wires or steel rebars. The panels being precast may have good precision in geometry and finishing, and structurally and economically efficient [2] and also have social and environmental benefits [3]. Information on precast technology can be found elsewhere [4–6]. These panels being lightweight have relatively less attraction of seismic forces, and also have advantages such as ease of handling and transportation.

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Peer review under responsibility of Faculty of Engineering, Alexandria University.

http://dx.doi.org/10.1016/j.aej.2016.11.016
1110-0168

Early in 1965, Pfeifer and Hanson [7] carried out number of tests to determine stiffness of small-scale concrete sandwich panels. Different types of shear connectors and core thickness were considered in their experimental study. They reported that, the amount and distribution of shear connectors significantly affect the flexural behavior of panels. They also noted that, the presence/absence of edge ribs affected the flexural behavior, and the failure load of the panels that had edge ribs was higher than the panels that did not have edge ribs. Pantelides et al. [8] have carried out experiments on concrete sandwich panels to determine the effect of using hybrid GFRP shell connectors on their flexural behavior and composite action. The experiments indicated that, the GFRP shell connectors provided resistance to horizontal shear between the concrete wythes and at the same time the panel could withstand out-of-plane loads. Bush and Wu [9] proposed equations to determine the deflection and flexural stresses of concrete sandwich panels. They found that the predicted deflection is comparable to the experimental deflection. Nevertheless they reported that, correction factors were required in order to get closer agreement with the experimental results.

Gara et al. [10] studied flexural behavior of precast concrete sandwich panels under four-point bending through experiments and numerical simulations. In their study, panels with different length and core thickness were considered. Wire mesh was used to reinforce wythes and non-shear connectors were used to connect the wythes. The authors reported that, all the panels tested behaved as semi-composite elements, and increasing panel thickness was necessary for increasing load carrying capacity. It was also noted that, provision of concrete beams at supporting edges was necessary to reduce longitudinal slip of wythes, and also to achieve higher bearing strength. Benayoune et al. [11] carried out experimental and theoretical studies on flexural behavior of precast concrete sandwich panels. Panels with three different sizes were tested. They used conventional rebars for wythe reinforcements and shear connectors. The authors reported that, the panels behaved as composite elements and the behavior was comparable to that of reinforced cement concrete (RC) slabs. Einea et al. [12] carried out experimental and analytical studies on flexural behavior of precast concrete sandwich panels with inclined Fiber Reinforced Polymer (FRP) bars as shear connectors. They reported that, the panel behavior was ductile, and the axial strength of the shear connectors governed the shear strength of the panels. Effect of truss connectors on the behavior of concrete sandwich panels was studied by Bush and Stine [13]. They reported that, high degree of composite stiffness and composite flexural strength may be obtained by using truss girders oriented longitudinally in the panels. They also reported that constructional details have significant effect on shear distribution in the elements crossing the interface. They also noted that, the insulation provided as the core offered significant shear resistance to the panel. Salmon et al. [14] showed that, use of FRP connectors improved thermal efficiency of panels compared to steel or concrete connectors. The ultimate load carrying capacities of the panels were found to be comparable to the panels that act as fully composite elements. They reported that, the actual stiffness of the panel was higher than the predicted stiffness. They also reported that the thermal efficiency of the panel using FRP connectors is nearly 1.75 times higher than that of using steel connectors.

Tomlinsons and Fam [15] examined the effect of adhesion and friction between concrete and insulation and reported that, adhesion and friction between concrete and EPS contributed to 44–59% of the ultimate load. Analytical model to predict shear stress of composite panels with truss shear connectors was proposed by Bush and Wu [9]. Recently, Adawi et al. [16] reported experimental studies to investigate composite action between machine-cast hollowcore slab and concrete topping. Tests were carried out to determine bond strength and shear strength of the panels. Their experimental studies also included testing prototype panels under three point bending with variations in panels based on panel thickness and discontinuity in concrete topping. Vertical deflection, slip between hollowcore and topping, and strain variations were monitored. The results indicated that, intentional surface roughness provided on hollowcore panels was higher than the machine cast finish. Bond and shear strengths were found to satisfy Canadian standard [17] and North American design standard, respectively. Flexural tests indicated that, hollowcore slabs with machine cast finish and acceptable roughness can provide composite action of the panels. They also conclude that slip and peel deformation did not affect the overall flexural behavior of the panels that may be attributed due to confining action provided by the load. Adawi et al. [18] also have provided analytical methods for determination of peel and shear stresses of panels cast with machine cast hollowcore and concrete topping. Structural engineers may use these methods to evaluate the peel and shear stresses for judging composite action of these types of panels. Literature survey indicates that research studies with respect to flexural behavior of concrete sandwich panel under punching load are not available in the literature.

2. Research significance

The present experimental study investigates effect of different types of loading conditions such as punching load and four-point bending on flexural behavior of precast concrete sandwich panels. The study also investigates possible effect of edge ribs provided along the supporting edges of panels on the flexural behavior of panels. The area of reinforcement in wythes and numbers of truss-like shear connectors are the same for the panels considered in this study. Also, predictability of linear elastic theory to determine the cracking moment of the panel is verified. The need for test results under different loading conditions for validating analytical and finite element models toward developing guidelines for design of precast concrete sandwich panels is the motivation for the present study.

3. Panel description and materials

Two prototype panels are cast and tested in the present study. The dimension (Length × Breadth × Thickness) of the panels tested under punching load and four-point bending was 1220 × 1220 × 150-mm and 3000 × 1220 × 150-mm, respectively. Pictures of typical EPS panel used and the schematic sketch of the components of a concrete sandwich panel used are shown in Figs. 1 and 2 respectively. On either side of EPS welded wire mesh of size 100 × 100-mm is used as wythes reinforcements. The wythes are connected using truss-like shear connectors that span along the longitudinal direction with their wires inclined at an angle of 45°. The wires of mesh

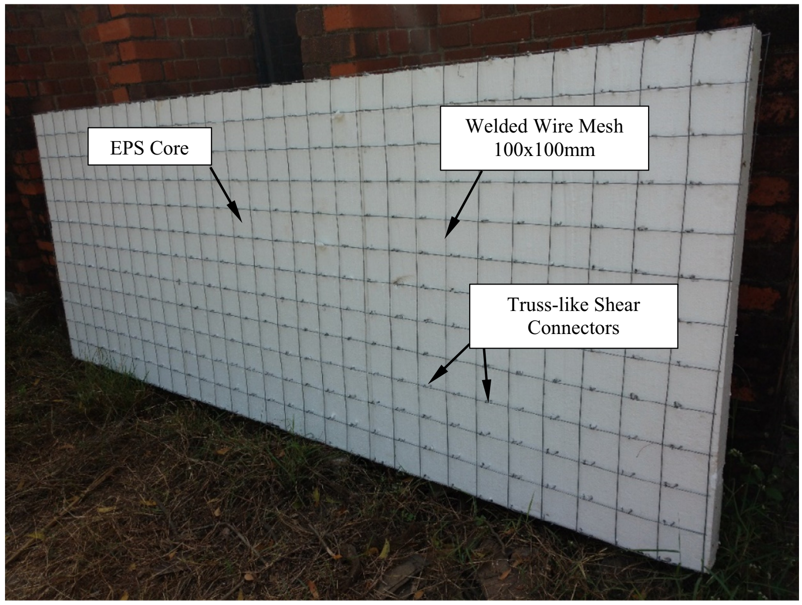

Figure 1 Typical EPS panel used in experimental study.

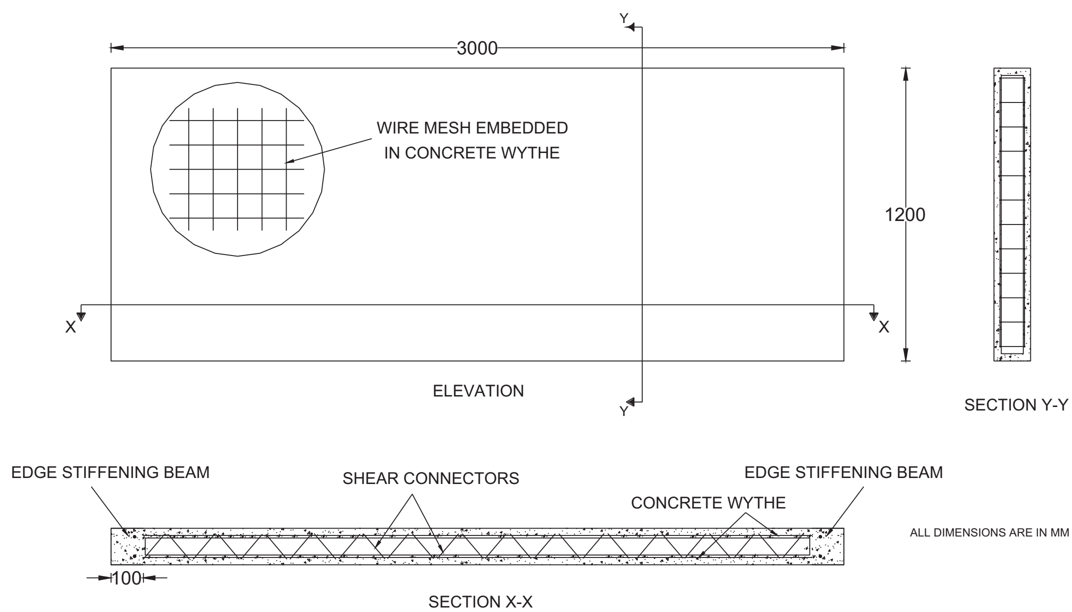

Figure 2 Schematic sketch of concrete sandwich panel.

and the shear connectors are nearly 2.2-mm in diameter with average material tensile strength of 651.64-MPa. The spacing of shear connectors was 100-mm and total number of truss-like shear connectors provided in each panel is 13. Self-Compacting Concrete (SCC) with a slump of >650-mm is used for casting the wythes. The mix proportion for SCC is arrived based on the guidelines of ACI 237R-07 [19] and it is 1:1.89:2.34:0.3:0.41:0.6% in the order of Cement, Coarse aggregate, Fine aggregate, Ground Granulated Blast Furnace Slag, Water and Superplasticizer (by weight of binder content). The total powder content used is 481-kg/m^3 and coarse aggregates passing 10-mm sieve are used. The SCC mixture satisfied the recommended minimum requirements [19]. Same mix proportion is used for casting both panels. Average cube compressive strength and tensile strength of SCC are determined to be 45.97-MPa and 4.34-MPa, respectively. The tensile strength is determined using concrete prism specimens of size $150 \times 150 \times 700$-mm as per IS [20].

4. Fabrication of panels

The sequence of casting a panel is shown in Fig. 3. A steel mold of required size is placed on a level surface and SCC is poured to a depth of 25-mm to form bottom wythe of the panel. EPS panel is placed on the concrete and SCC is poured to form top wythe of 25-mm thickness. To study the possible

Figure 3 Sequence of casting a panel.

effect of presence/absence of edge rib, for the panel specimen subjected to four-point bending concrete edge rib is provided along the supporting edges. The panels cast are cured for 28 days.

5. Test setup and instrumentation

The panels are tested to failure under static loading. Load is applied by 100-kN hydraulic jack. Linear Voltage Displacement Transducers (LVDTs) with 50-mm range are used to measure transverse deflections. Strain gauges with gauge length of 30-mm are used to measure strains on wythe surfaces. One edge of the panel is supported by a hinge and the other is supported by a roller. Schematic sketch of the test setups used and instrumentation is shown in Figs. 4 and 5. Photographs of panels ready for testing are shown in Figs. 6 and 7. First crack loads reported are the loads at which first crack is visually seen on the sides or soffit of the panel.

6. Test results and discussions

Summary of test results is given in Table 1. Cracking moment, principal tensile stress at extreme bottom fiber of the panel and ultimate moment are calculated at loading point using first principles.

Load-deflection curves obtained from the experiments and the corresponding moment-deflection curves are shown together in Fig. 8.

Results and discussions pertaining to punching load test are presented first followed by the results and discussions with respect to four-point bending test. In the punching load test, first crack occurred below the loading point in bottom wythe of the panel at a load of 23.2-kN that corresponds to a bending moment of 5.9-kN m. Cracks in top wythe occurred at a load of 27.5-kN. Horizontal cracks at the EPS-to-concrete interface occurred at supporting edges after cracks occurred in top wythe. Cracks seen are shown in Figs. 9a–9c.

Number of flexural cracks occurred in bottom wythe, and hence the truss-like shear connectors are effective to achieve composite action of the panel until failure. Inclined shear cracks are also seen on the sides of the panel that indicated classical flexural behavior of the panel. It is reported that conventional RC Slabs subjected to punching load are liable to fail by forming radial yield line patterns [21] (circular fans – see Fig. 10).

The crack pattern in the bottom wythe of the panel is thus found to be similar to that expected in a square RC slab subjected to a punching load. Therefore, it may be concluded that the flexural behavior of precast concrete sandwich panel considered in this study under punching load is similar to conventional RC slab. Horizontal cracks at EPS-to-concrete interface are attributed to the absence of edge ribs at supporting edges.

Fig. 8 indicates that the panel behaved linearly up to a load of 17.5-kN beyond which the behavior is nonlinear until failure. Bending moment corresponding to this load at the loading point is 4.5-kN m, which is approximately 75% of cracking moment. A fall in the load-deflection curve is seen at the first crack load. After cracks occurred in top wythe, there is no appreciable increase in the load. The strain variations measured in the top wythe are shown in Fig. 11. The strains at C1 and C3 are tensile in nature and strains at C2 are compressive. Therefore, the stress in top wythe is compressive under the loading point and it is tensile at distances away from it. This indicates that, the panel bending is not cylindrical, because in cylindrical bending of plate (made of homogenous and isotropic material) only compressive stress occurs in extreme top fibers. The reason for this may be attributed to possible frictional resistance provided by supporting plates that might provide partial moment restraints at the supported edges. The strains at C1 and C3 increased at faster rate after cracks occurred in top wythe. Higher strain values at C2 could

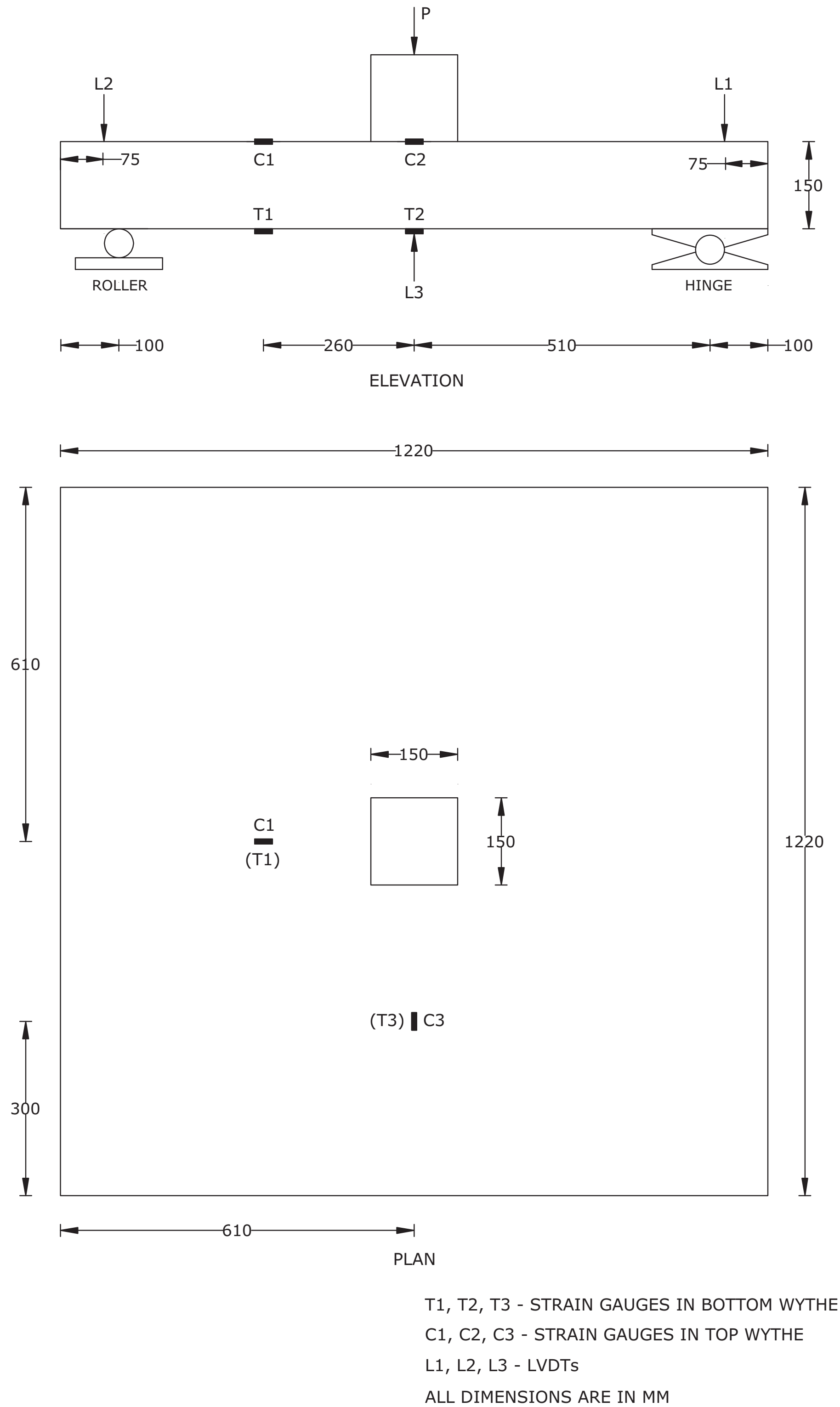

Figure 4 Test setup and instrumentation details for punching load test.

be achieved due to compressive stress than at C1 and C3 which are tensile in nature.

The strain variations at C2 may be considered to be linear as long as the panel behavior is linear. But the strain variations at C1 and C3 are linear only up to a load of 13-kN, which may be due to tensile nature of stress. The strain reversal at C2 beyond 30-kN is attributed due to widening of tensile cracks in top wythe that relieved strain at C2. Fig. 12 shows strain variations measured in bottom wythe. The stress in bottom wythe is tensile in nature as expected. Reversal of strains is attributed to formation of cracks at other locations in bottom

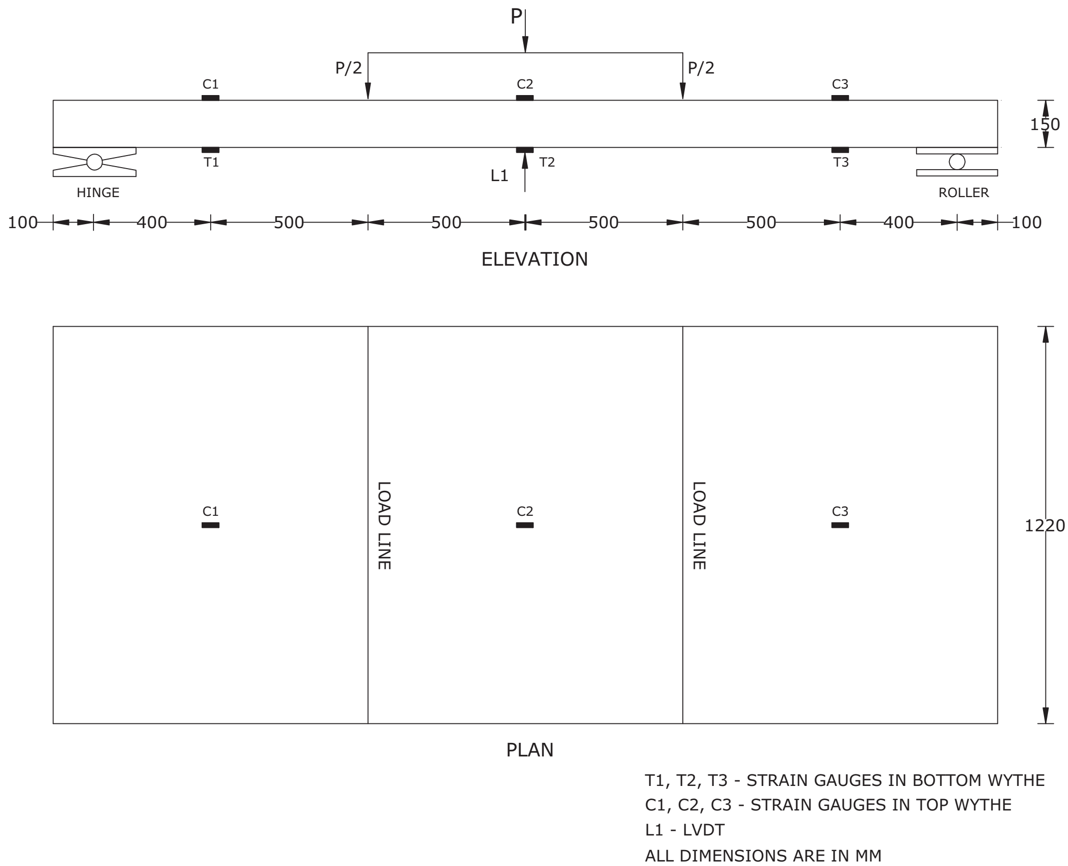

Figure 5 Test setup and instrumentation details for four-point bending test.

Figure 6 Photograph of panel ready for testing under punching load.

wythe that relieved the stress, and hence the strain, at instrumented locations.

In four-point bending test, first crack occurred in bottom wythe at a load of 12.5-kN. Crack was visible on the sides of the panel at 14.6-kN, and at that load the panel failed with a breaking sound at a cross section located in constant shear zone approximately 30-mm away from one of the loading points. The tested panel is shown in Fig. 13. Unlike the panel tested under punching load, more number of cracks did not occur in the bottom wythe. Failure occurred at a cross section in the constant shear zone region which is also close to maximum bending moment region. Also, cracks in shear region were wider than those in maximum bending moment region. From these observations, the panel failure may be attributed to failure of concrete in bottom wythe due to combined effect of shear and flexural stresses. Wythe separation, that is evident

Figure 7 Photograph of panel ready for testing under four-point bending.

Table 1 Summary of test results.

S. No.	Loading condition	First crack load (kN)	Cracking moment (kN m)	Calculated principal tensile stress at first crack load (N/mm^2)	Ultimate load (kN)	Ultimate moment (kN m)	Moment up to which panel behaved linearly (kN m)	Remarks
1	Punching	23.2	5.90	1.9	31.9	8.1	4.5	Flexural mode of failure
2	Four-point bending	12.5	5.70	1.8	14.6	6.6	5.6	Combined effect of shear and flexural stress caused panel failure

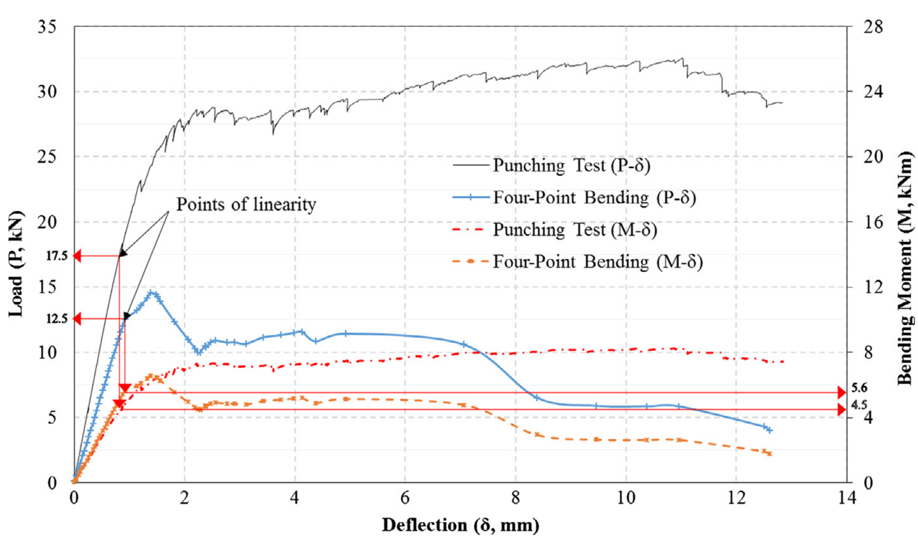

Figure 8 Load-deflection and moment-deflection curves.

from horizontal cracks at concrete-to-EPS interface did not occur, and hence the panel may be considered to have behaved as composite element until failure.

Fig. 8 indicates that panel behavior was linear up to 12.5-kN beyond which it is nonlinear until failure. There is a change in slope of the load-deflection curve after this load, and is attributed to formation of first crack and subsequent minor cracks in bottom wythe that caused reduction in the stiffness. Bending moment corresponding to this load at loading point is 5.7-kN m. One of the cracks widened with a very small percentage (≈16%) increase in the load to cause the failure with breaking sound.

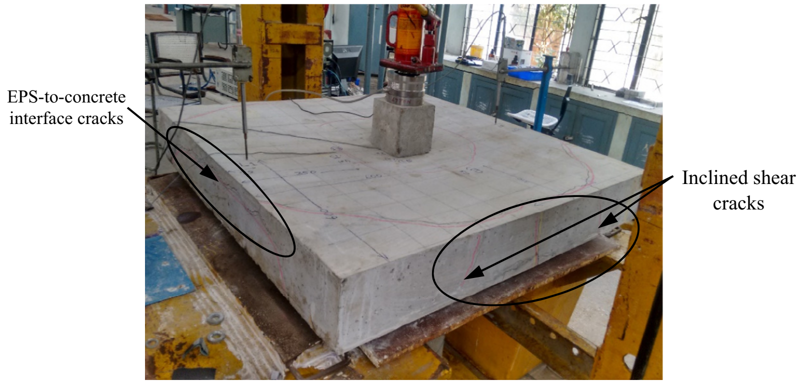

Figure 9a Panel tested under punching load.

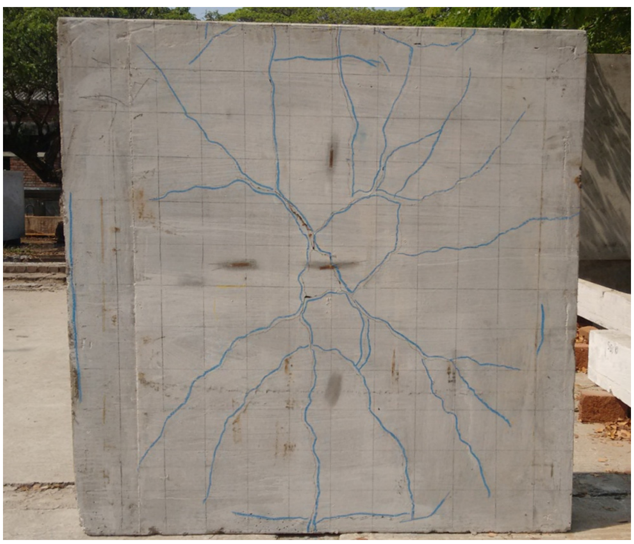

Figure 9b Cracks seen in bottom wythe (under punching load).

Figure 9c Cracks seen in top wythe (under punching load).

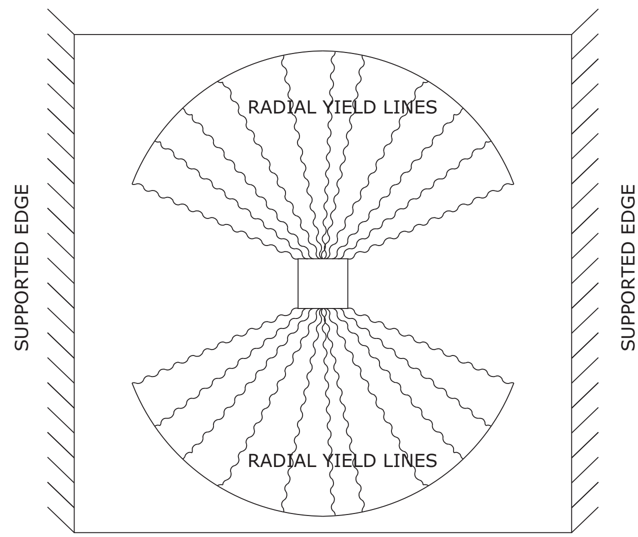

Figure 10 Yielding patterns under concentrated load [21].

The strain variations measured in wythes of the panel are shown in Figs. 14 and 15. The nature of strain values indicates that top wythe is subjected to compressive stress and the bottom wythe is subjected to tensile stress. The magnitude of strains measured at C2 is relatively more than those at C1 and C3. These observations indicate that, the panel bending may be considered to be cylindrical at least in constant bending moment region up to the point of linearity. After first crack load, there is reduction in strain values measured. This is attributed to widening of crack in bottom wythe which relieved stress in top wythe.

For the two different types of loading conditions considered, calculated cracking moment is nearly the same for both panels. The principal tensile stress values calculated are much lower than the tensile strength of the concrete. This may be due to the presence of shrinkage stresses, redistribution of shear stresses between flexural stresses and local weakening of the cross section by transverse reinforcement [22].

Fig. 8 indicates that, moment-deflection curves of the panels tested under different loading conditions are nearly the same up to a bending moment of 4-kNm. It is noted that irrespective of loading conditions, beyond 6.8-kNm (average) there is significant degradation in stiffness and strength of the panels. In general, the test results indicate that flexural

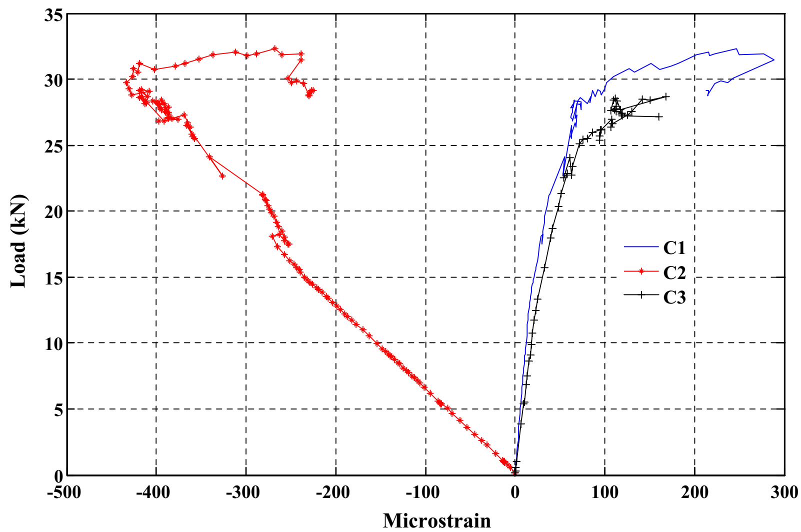

Figure 11 Strains measured in top wythe (punching load test).

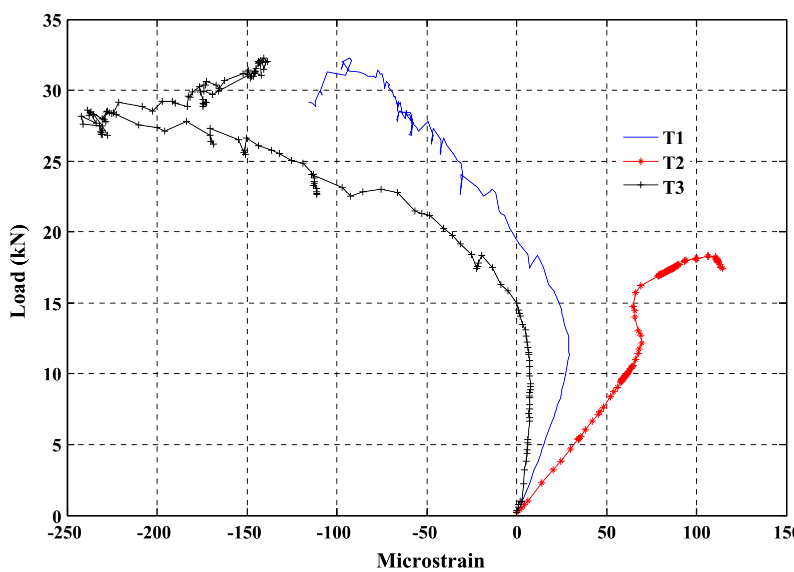

Figure 12 Strains measured in bottom wythe (punching load test).

behavior of concrete sandwich panels is significantly affected by type of loading conditions. Maximum bending moment resisted by both panels is nearly equal, and only post peak behavior of M-δ curve differs. Under punching load better energy dissipation is achieved by formation of number of flexural cracks resulting in flexural and ductile mode of failure, whereas sudden failure occurred under four-point bending. The maximum strain values measured in wythes in four-point bending test are lower than those measured in punching load test. This indicates that welded wire mesh (as reinforcement) provided in bottom wythe is insufficient to redistribute the stresses after formation of first crack. If the bottom wythe of the panel is reinforced by providing additional conventional steel rebars to redistribute the stresses after formation of first crack, the ultimate load carrying capacity of the panel under four-point bending may increase.

7. Analytical study

In this section, based on linear elastic theory, deflection, stiffness and cracking moment of the panels are calculated. The predictions and discussions are restricted only up to linear behavior of the panels. In the calculations, wythes are assumed to resist bending and shear connectors are assumed to resist shear. Deflection due to shear and possible contribution of EPS to resist bending and shear stresses are not considered in the calculations. Also, wire mesh is not considered in determining the bending strength of the cross section. Table 2 shows the elastic cross-sectional properties of the panel (see Fig. 16). The formulae used for calculating the deflections may be found elsewhere [23,24].

The results of analytical predictions are given in Table 3. The stiffness of panel subjected to punching load based on lin-

Figure 13 Crack seen in side and bottom of the panel (four-point bending test).

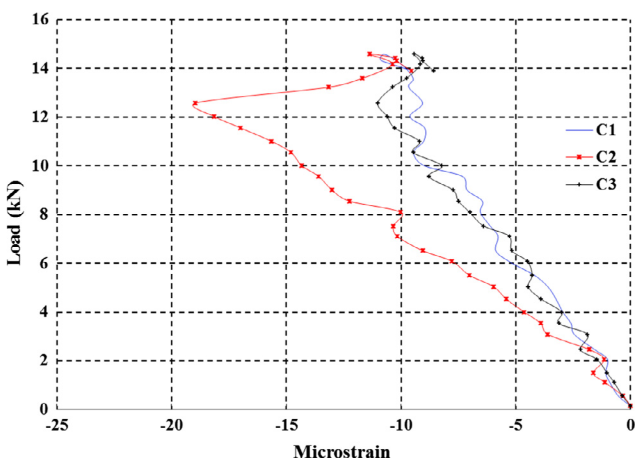

Figure 14 Strain variations measured in top wythe (four-point bending test).

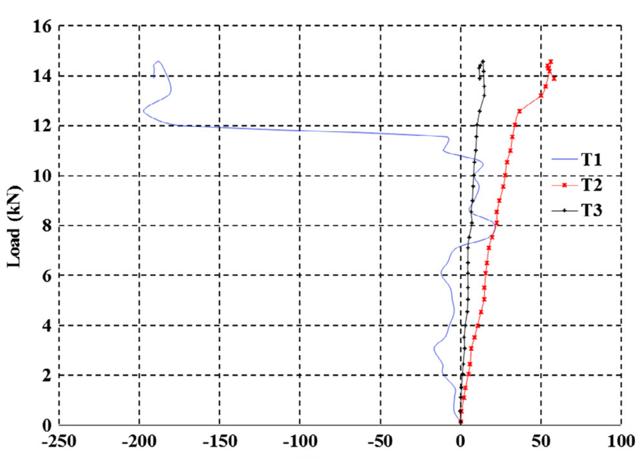

Figure 15 Strain variations measured in bottom wythe (four-point bending test).

ear elastic analysis is more than the actual stiffness of panel, and hence the predicted deflection is very low compared to actual deflection. The predicted stiffness of the panel subjected

Table 2 Cross-sectional and material properties.

Properties	Magnitude
Width of panel, b	1220 mm
Thickness of wythe, t	25 mm
Thickness of panel, h	150 mm
Center-to-center distance of wythes, d	125 mm
Moment of inertia (neglecting core)	238.3×10^6 mm^4
Elastic section modulus, Z	3.18×10^6 mm^3
Young's modulus [25], E	33541.0 N/mm^2
Tensile strength of concrete [25], f_t	4.7 N/mm^2
Flexural rigidity, EI	7.99×10^{12} Nmm^2

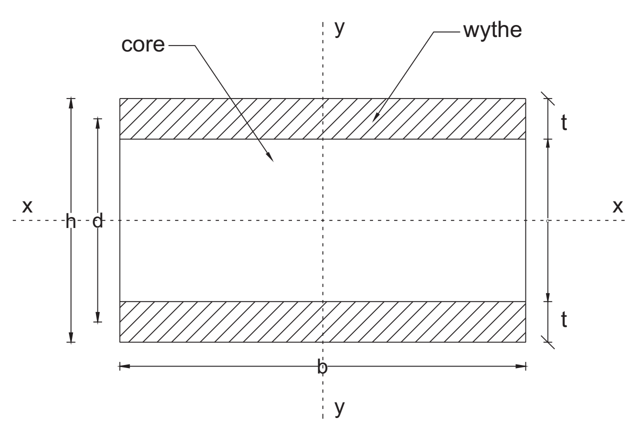

Figure 16 Cross-sectional details.

to four-point bending is comparable to its actual stiffness, and hence the measured and actual panel deflections are comparable. The actual stiffness of the panel subjected to four-point bending is lower than the actual stiffness of the panel that is subjected to punching load. This is attributed due to larger dimension of the panel subjected to four-point bending.

The cracking moment of the panel is determined using average principal tensile stress of 1.85-N/mm^2. The cracking moment predicted is comparable to the cracking moment of the panels determined from experiments.

Table 3 Results of Analytical Predictions.

S. No.	Panel	Loading type	Load (kN)	Actual deflection (mm)	Predicted deflection (mm)	Actual stiffness (kN/mm)	Predicted stiffness (kN/mm)	Predicted cracking moment, m_u (kNm)
1	Square	Punching	17.5	0.86	0.05	20.3	361.5	5.9
2	Rectangular	Four-Point Bending	12.5	0.93	0.86	13.4	14.5	5.9

Experimental study indicated that failure of the panel subjected to four-point bending is sudden and failure is brittle without forming number of flexural yield lines. And hence, yield line theory may not be applicable to predict its ultimate load carrying capacity. The panel specimen subjected to punching load failed by forming number of flexural yield lines and hence equation [21] based on yield line theory for conventional RC slab may be used to predict the ultimate load. The failure load corresponding to formation of radial yielding pattern is given by [21],

$$P_u = 2\pi m_u = 6.28 m_u \tag{1}$$

where m_u ($= f_t Z$) is the bending strength of the cross section (Cracking moment).

Ultimate load, $P_u = 6.28 \times 6.20 = 38.9$-kN.

The ultimate load predicted using yield line theory is nearly 20% more than the experimental ultimate load. Statistical variation in the material and cross-sectional properties may be some of the reasons for this reduction. Eq. (1) may be used to determine the average principal tensile strength of concrete by equating to experimental ultimate load as follows.

$$31.96 \times 10^3 = 6.28 \times f_t \times Z$$

$$31.96 \times 10^3 = 6.28 \times f_t \times 3.18 \times 10^6$$

$$f_t = 1.60\ \mathrm{N/mm^2}$$

This is nearly 15% lower than the average principal tensile stress calculated corresponding to first crack load, and is much lower than the tensile strength of the concrete. The tensile strength of concrete is seen to affect the analytical prediction of cracking moment and the ultimate load of concrete sandwich panels significantly. The tensile strength of concrete is reported [26] to also play a vital role in affecting load-deflection curves of concrete sandwich panels obtained from numerical simulations developed in finite element package ABAQUS [27].

Since the panel subjected to punching load did not fail due to punching or two-way shear, the shear strength of the panel provided by truss-like shear connectors (neglecting wythes) at any cross section of the panel is calculated using ACI 318 [28]. It is assumed that the connection between shear connectors and welded wire mesh did not fail. ACI 318 [28] permits the use of wires of wire mesh oriented perpendicular to longitudinal reinforcement as shear reinforcement, but the wires are inclined in the panels considered in the experimental study. It also restricts the maximum tensile strength of wires used as shear connectors to 551.6-MPa wherein the yield strength of the wires used in the present study is nearly 20% higher than this limit. Shear strength provided by inclined shear reinforcement may be determined using ACI 318 [28] equation as follows.

$$V_s = \frac{A_v f_{yt}(\mathrm{Sin}\,\alpha + \mathrm{Cos}\,\alpha)d}{s}$$

Number of shear connectors = 13

A_v – Area of shear connector = 13 × 3.8 = 7.6-mm^2
f_{yt} – Yield strength of shear connector wire = 651.6-N/mm^2
α – Angle of inclination of shear connector = 45°
d – Depth of member from extreme compression fiber to mid of longitudinal reinforcement = 135-mm
s – Distance measured in direction parallel to longitudinal reinforcement = 100-mm

On substitution, $V_s = 61.8$-kN. The predicted shear strength provided by shear connectors is higher than the ultimate load of the panel. This may be the reason for flexural failure of the panel. There seems to be promising future for using EPS panels with continuous truss-like shear connectors for construction of flat plates and flat slabs which have to be verified with further experimental and analytical studies. Experimental and numerical studies are essential in this area to identify possible failure modes, and to determine ultimate flexural load carrying capacity of concrete sandwich panels under different loading and support conditions.

8. Summary and conclusions

Results of experimental and analytical studies carried out to understand and compare the effect of different loading conditions such as punching load and four-point bending on flexural behavior of concrete sandwich panels are presented and discussed. The flexural behavior of panel subjected to punching load considered in this study is similar to conventional RC slab. Unlike conventional RC slab, punching or two-way shear did not govern panel failure. The failure of the panel subjected to four-point bending is sudden, and crack initiation and propagation that caused panel failure may be attributed due to combined effect of flexural and shear stresses. Actual deflection of the panel subjected to four-point bending is comparable to the deflection predicted, and the ultimate load based on yield line theory is comparable to the experimental ultimate load of the panel subjected to punching load. Cracking moment predicted is comparable to experimental cracking load for both types of loading conditions considered. Experimental study in general indicates that, the type of loading conditions significantly affects flexural behavior of concrete sandwich panels. Also, there seems to be promising future for use of precast concrete sandwich panels for flat plates and flat slabs. Experimental and numerical studies to understand the behavior of precast lightweight concrete sandwich panels under different types of loading and support conditions are required in this area for developing design guidelines for practical applications.

References

[1] A. Einea, D.C. Salmon, G.J. Fogarasi, T.D. Culp, M.K. Tadros, State-of-the-art precast concrete sandwich panels, PCI J. 36 (6) (1991) 78–98.

[2] A.A. Yee, Structural and economic benefits of precast/prestressed concrete construction, PCI J. 46 (4) (2001) 34–42.

[3] A.A. Yee, Social and environmental benefits of precast/prestressed concrete construction, PCI J. 46 (3) (2001) 14–19.

[4] A.A. Yee, Prestressed concrete for buildings, PCI J. 24 (5) (1976) 112–157.

[5] A.A. Yee, One hundred Washington square: structural design and construction, PCI J. 29 (1) (1984) 24–48.

[6] A.A. Yee, Design considerations for precast prestressed concrete building structures in seismic areas, PCI J. 36 (3) (1991) 40–55.

[7] D.W. Pfeifer, J.A. Hanson, Precast concrete wall panels: flexural stiffness of sandwich panels, ACI Spec. Publ. 11 (1965) 67–86.

[8] C.P. Pantelides, R. Surapaneni, L.D. Reaveley, Structural performance of hybrid GFRP/Steel concrete sandwich panels, J. Compos. Constr. 12 (5) (2008) 570–576.

[9] T.D. Bush, Z. Wu, Flexural analysis of prestressed concrete sandwich panels with truss connectors, PCI J. 43 (5) (1998) 76–86.

[10] F. Gara, L. Ragni, D. Roia, L. Dezi, Experimental tests and numerical analysis of floor sandwich panels, Eng. Struct. 36 (2012) 258–260.

[11] A. Benayoune, A.A.A. Samad, D.N. Trikha, A.A. Abang Ali, S. H.M. Ellinna, Flexural behavior of pre-cast concrete sandwich composite panel – experimental and theoretical investigations, Constr. Build. Mater. 22 (2008) 580–592.

[12] A. Einea, D.C. Salmon, M.K. Tadros, T. Culp, A new structurally and thermally efficient precast sandwich panel system, PCI J. 39 (4) (1994) 90–101.

[13] T.D. Bush, G.L. Stine, Flexural behaviour of composite precast concrete sandwich panels with continuous truss connectors, PCI J. 39 (2) (1994) 112–121.

[14] D.C. Salmon, A. Einea, M.K. Tadros, T.D. Culp, Full scale testing of precast concrete sandwich panels, ACI Struct. J. 94 (4) (1997) 354–362.

[15] D. Tomlinson, A. Fam, Experimental investigation of precast concrete insulated sandwich panels with glass fiber-reinforced polymer shear connectors, ACI Struct. J. 111 (3) (2014) 595–606.

[16] A. Adawi, M.A. Youssef, M.M. Meshaly, Experimental investigation of the composite action between hollowcore slabs with maching-cast finish and concrete topping, Eng. Struct. 91 (2015) 1–15.

[17] CSA A23.2, Test Methods and Standard Practices for Concrete, Canadian Standard Association (CSA), Mississauga, Canada, 2009.

[18] A. Adawi, M.A. Youssef, M.M. Meshaly, Analytical modeling of the interface between lightly roughened hollowcore slabs and cast-in-place concrete topping, ASCE J. Struct. Eng. 141 (4) (2015) 1–9.

[19] ACI 237R-07, Self-Consolidating Concrete, 2007.

[20] IS 516:1959, Methods of Tests for Strength of Concrete, Bureau of Indian Standards, 2002.

[21] R. Park, W.L. Gamble, Reinforced Concrete Slabs, John Wiley & Sons, Inc., New York, 1980.

[22] R. Park, T. Paulay, Reinforced Concrete Structures, John Wiley & Sons Inc., 1975.

[23] WEB. <http://ocw.mit.edu/courses/materials-science-and-engineering/3-11-mechanics-of-materials-fall-1999/modules/bdisp.pdf>.

[24] R.C. Hibeller, Structural Analysis, eighth ed., Prentice Hall, 2012.

[25] IS456:2000, Plain and Reinforced Concrete – Code of Practice. Bureau of Indian Standards, 2005.

[26] J. Daniel Ronald Joseph, J. Prabakar, P. Alagusundaramoorthy, Experimental and numerical study on flexural behavior of precast light-weight concrete sandwich panels, Int. J. Res. Eng. Technol. 4 (13) (2015) 463–467.

[27] ABAQUS/Standard User's Manual for Version 6.10.

[28] ACI Committee 318. Building Code Requirements of Structural Concrete, 2011.