UNB | Off-site Construction Research Centre
UNIVERSITY OF NEW BRUNSWICK EST. 1785

Northumbria University NEWCASTLE

Transforming Construction with Off-site Methods and Technologies (TCOT) Conference: Designing Tomorrow's Construction, Today

August 20-22, 2024, Fredericton, New Brunswick, Canada

STRUCTURAL PERFORMANCE OF LIGHT-WEIGHT PRECAST CONCRETE WALL PANELS

McNeill, N[1,*], Lloyd, A[2]
[1,2] Department of Civil Engineering, University of New Brunswick, Canada [*]
nathan.mcneill@unb.ca

Abstract: Precast concrete is a method of construction involving the use of concrete structural members that have been formed, mixed, and cured off site. The advantages of precast concrete construction are numerous including: consistency, better vibration procedures, increased speed of construction, controlled curing, and enhanced quality control. One of the more common structural components made from precast concrete are wall panels. This research is focused on the design, construction, and testing of light-weight precast concrete wall panels against hurricane loading for housing applications. For this study two methods were used to reduce the overall wall panel weight: designing the walls as waffle panels to minimize volume and replacing 30% of the volume of concrete with expanded polystyrene (EPS) beads to minimize concrete self-weight. While each of these methods of creating light-weight concrete are well researched and documented on their own, little research has been completed on the performance of concrete wall panels when combining the two. To validate the efficacy of the wall panels, a testing program was devised to test the wall panels under flexural and combined axial and flexural loading. Eight concrete wall panels were constructed and tested until failure at the University of New Brunswick in Fredericton. Load cells and spring extensometers were used to track the loads and deflections during testing. This paper will explore the results of the testing and draw conclusions on the constructability and adequacy of these wall panels.

Keywords: Precast; Light-weight; Concrete; Waffle Panels; Expanded Polystyrene (EPS)

1 INTROCUCTION

The main design objective of this research was to design a precast, lightweight, easy to construct, concrete wall panel that can reach a sufficient strength to withstand category five hurricane wind speeds (160 mph) in accordance with CSA A23.3-14 Design of Concrete Structures and with the National Building Code of Canada (NBCC 2015). Another design consideration was having the concrete reach a sufficient strength after three hours so that it could be demoulded and stored to continue curing.

Other than the strength of the wall panels, the most important design consideration was the overall weight of the panels. Two methods were incorporated into the design of the wall panels to reduce their overall weight: a waffle panel design to minimize volume and replacing 30% of the volume of concrete with expanded polystyrene (EPS) beads for an approximate 30% material weight reduction. Waffle panel designs typically have the reinforcing steel in a grid pattern and the areas between reinforcing have a thinner concrete thickness. This creates the “waffle” pattern design which reduces the overall weight of the panel. To create the voids in the concrete between lines of reinforcing rigid insulation was used in the forms (Figure 1).

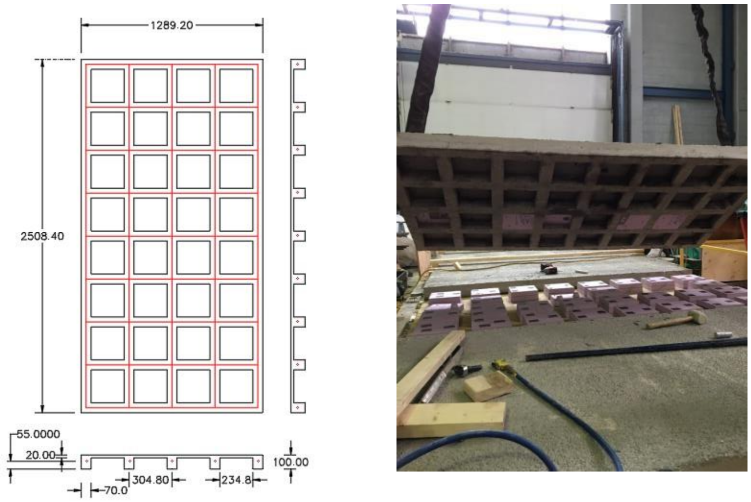


Figure 1: Dimensions of Waffle Panel

Ten concrete panels were constructed and tested in pure flexure, pure axial, and combined axial and flexural loading in accordance with ASTM E72: Standard Test Methods of Conducting Strength Tests of Panels for Building Construction (ASTM 2022). The flexural and axial loads were applied using Enerpac hydraulic jacks and the loads and deflections were measured using load cells and string potentiometers, respectively. The tests performed under pure axial load will not be included in this paper as this paper focuses on the efficacy of the panels against out-of-plane (wind) loading.

Conclusions on the efficacy of these wall panels as a building material capable of withstanding category five wind loading were drawn based on the results of the testing and will be discussed below.

2 LITERATURE REVIEW

2.1 Precast Concrete Wall Panels

Precast concrete construction is a method of construction involving the use of concrete structural members that have been formed, mixed, and cured off site. From there, the final product is transported to the construction site and assembled (Seifi et al. 2019). Some of the advantages of precast concrete include consistency, strength, better vibration procedures, increased speed of construction and controlled curing (Tomek 2017, Seifi et al. 2019). Precast concrete also has the ability to be pretensioned during production and post-tensioned during installation. One of the more common products made from precast concrete are precast concrete wall panels. Precast concrete wall panels are extremely versatile structural components that can be designed in many unique ways to meet the needs of almost any project. One of the most common uses of precast wall panels are as main load bearing elements in buildings (Seifi et al. 2019). Precast wall panels can be designed to withstand in and out of plane forces as well as axial loads. There are multiple types of wall panels including plain concrete wall panels, insulated concrete wall panels, hollow core concrete wall panels, and double tee concrete wall panels (Freedman 2019, Joseph 2020, Nasser et al. 2015, Xiong et al. 2018) Insulated concrete wall panels and waffle panels will be explored further below.

2.1.1 Lightweight Precast Concrete Sandwich Panels

Precast concrete sandwich panels are an alternative to typical concrete wall panel construction. These wall panels are constructed using two sections of reinforced concrete on either side of a lightweight insulating material connected using either concrete webs, metal connectors, plastic or composite connectors, or a combination as shear connectors (Einea et al. 1991, Joseph 2020). There are many advantages to these panels, including low weight, thermal efficiency, acoustic performance, and ease of construction (Einea et al. 1991) With the most relevant advantages to this paper being the lightweight and thermal efficiency.

2.1.2 Concrete Waffle Panels

Waffle panels, also known as waffle slabs, or two way ribbed flat slabs, are a type of lightweight panel being used in modern construction. These panels consist of multiple reinforced concrete joists running orthogonally to each other to create a grid. The panel thickness between the lines of reinforcement, or joists is able to be minimized due to the relatively short spans between consecutive joists. The reduction in thickness between the concrete joists enables this type of panel to have a lightweight cross section. These panels are typically used as slabs in floor systems however, they can also be used as the main structural element in wall systems and as bridge deck elements (Aaleti et al. 2011, Abdul-Wahab & Khalil 2000).

2.2 Lightweight Concrete

Lightweight concrete can be created in a number of ways including, replacing conventional aggregates with either natural or artificial lightweight aggregates, aerating the concrete mixture to incorporate air voids, and eliminating the fine aggregates in the concrete to reduce the density (Jiang et al. 2016). This literature review will focus on lightweight concrete made by including artificial lightweight aggregates, namely, expanded polystyrene (EPS).

2.2.1 EPS Concrete

Expanded polystyrene (EPS) concrete is a lightweight concrete made by substituting proportions of coarse aggregates with EPS beads to produce a concrete with a range of densities. Low density is not the only attractive feature of EPS concrete, thermal insulation, acoustic insulation, low cost, and eco-friendly impacts help make EPS concrete a viable construction material (Nikbin & Golshekan 2018). Although EPS concrete has many favourable qualities, there are limitations when using this EPS as aggregate. Due to the ultralightweight of EPS beads and its hydrophobicity, workability can be a primary concern (Cook 1973). Furthermore, research has shown that with a higher percentage of EPS used in a concrete mix, many important mechanical properties of the concrete are negatively affected such as: compressive strength, splitting tensile strength, flexural strength, and modulus of elasticity (Alqahtani et al. 2017).

3 EXPERIMENTAL PROGRAM

All ten panels were constructed on site at the University of New Brunswick in Fredericton. Due to space constraints, two pours were completed creating five panels each pour. A method of vibratory compaction was devised to ensure the concrete would consolidate evenly throughout the panels. Due to the large area needed to cast five wall panels, the dip vibration method was deemed inappropriate as it would be difficult to ensure vibratory compaction had taken place throughout the panels. Instead, a shaker table was designed that would hold the concrete forms and vibrate them once the concrete had been poured (Figure 2). A dynamic actuator set to vibrate the concrete with a frequency of 15 Hz and an amplitude of 1 mm was used to vibrate and properly consolidate the concrete.

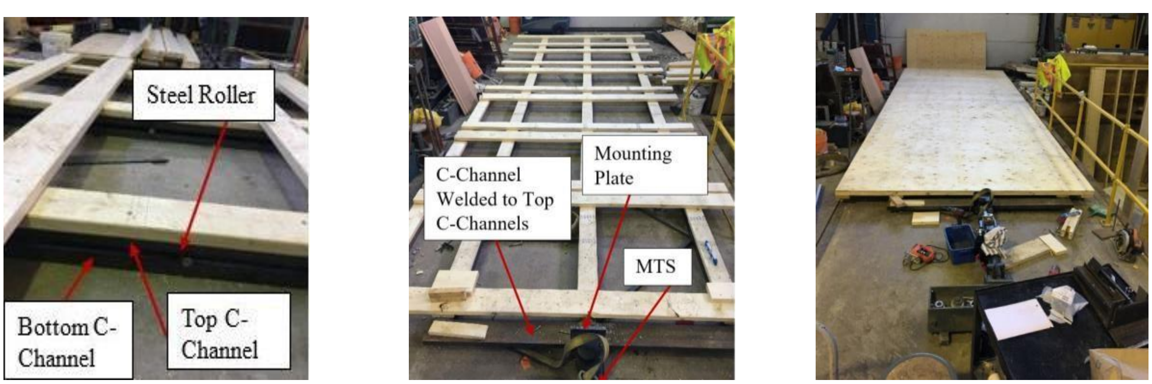


Figure 2: Shaker Table Construction

The forms were constructed using dimensional lumber for the sides and rigid insulation (high-density XPS sheets) to create the voids between the reinforcing in the concrete. Two sheets of rigid insulation were attached to the top of the shaker table and cut to size using a circular saw. The reinforcing grids were created by welding 10M reinforcing steel at the proper spacing. Small lengths of left-over reinforcing steel were used as rebar chairs and attached to the bottom of the grids so they would sit at the proper height in the forms. The shaker table, forms and reinforcing steel just prior to the first concrete pour can be seen in Figure 3.

Figure 3: Shaker Table and Forms Prior to Concrete Pour

To test the panels in pure flexure and combined axial and flexural loading in accordance with ASTM E72, an apparatus was constructed at the University of New Brunswick that made use of a large testing frame bolted to a reinforced concrete strong floor (Figure 4). This frame was used to hold the hydraulic used to load the panels in flexure.

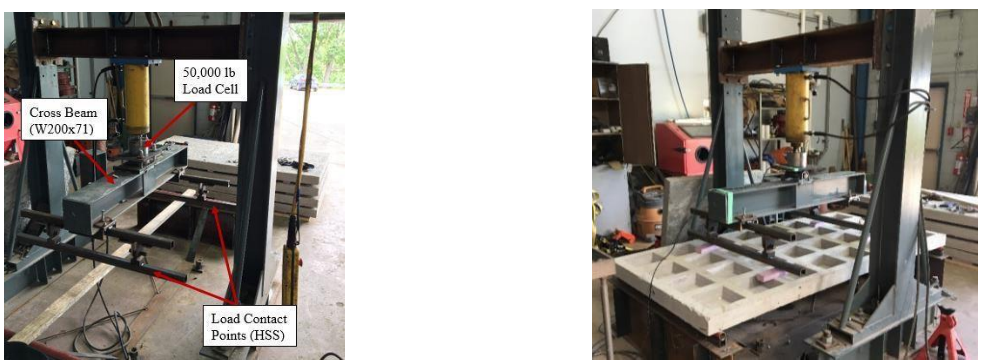


Figure 4: Testing Apparatus Used for Panels Under Pure Flexure

A self-reacting frame comprised of 1 ½” threaded rods and heavy steel W sections transferred axial load through hydraulic rams into the wall as shown in Figure 5.

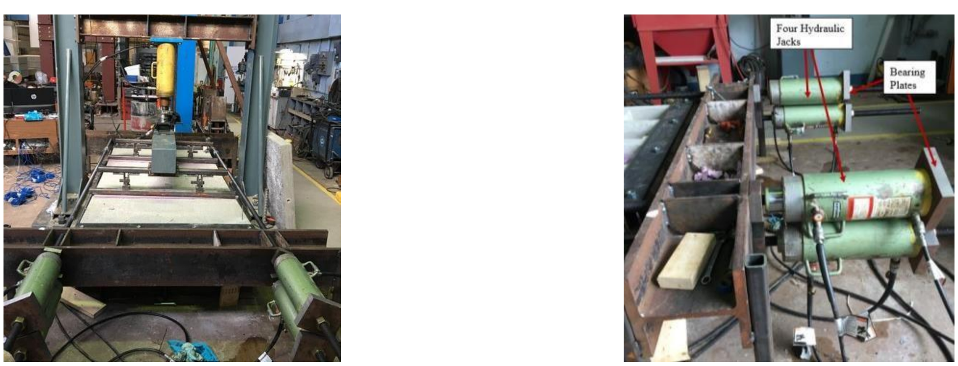


Figure 5: Testing Apparatus for Panels Under Combined Flexure and Axial Compression

The testing apparatuses described above were used to test all ten panels. The results of the testing will be discussed and shown graphically later in the next section of this paper.

4 EXPERIMENTAL RESULTS

4.1 Introduction

The experimental program was comprised of ten tests to be performed on the ten concrete wall panels. The program tested the panels under flexural, combined axial and flexural, and pure axial loading. Where this paper is focused on the efficacy of the wall panels under hurricane wind loads the results of the testing under pure axial load will not be included. As previously mentioned, two concrete pours were completed to construct the ten concrete wall panels. The four types of loading considered in this paper are: positive flexure, negative flexure, combined axial compression and positive flexure, and combined axial compression and negative flexure. The tests performed on the wall panels are summarized in Table 1 below.

Table 1: Summary of Testing Program

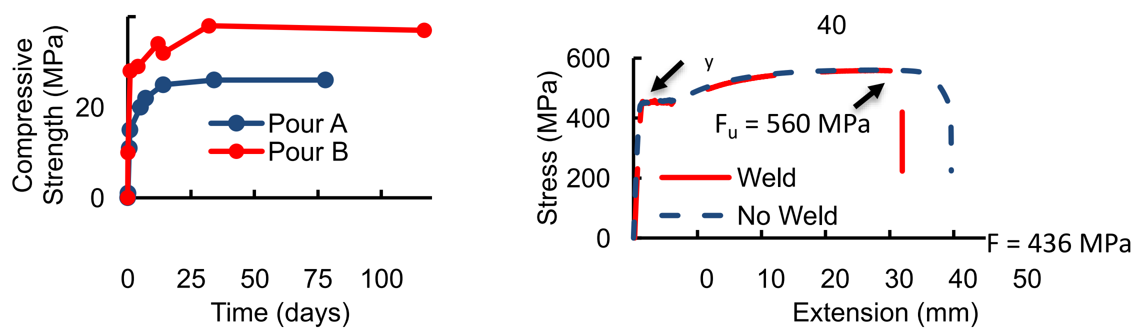
Test #	Concrete Pour	Panel Name	Positive Flexure	Negative Flexure	Axial Compression
1	A	A-PF		-	-
2	B	B-PF		-	-
3	A	A-NF	-		-
4	B	B-NF	-		-
5	A	A-A-PF		-	
6	B	B-A-PF		-	
7	A	A-A-NF	-		
8	B	B-A-NF	-		
*9	A	A-A	-	-	
*10	B	B-A	-	-	

*Test results not included in this paper

4.2 Material Properties

The compressive strength of each concrete pour was tested multiple times from three hours after casting until the day the wall panels were being tested. The panels were designed assuming a compressive strength (f'c) of 30 MPa. At the time of testing pour A & B had average compressive strengths of 26 MPa & 37 MPa, respectively. Pour A did not reach the design strength of 30 MPa due to extra water being added to the concrete mix that was not accounted for. Aggregates used in the concrete mix for pour A were covered in snow and added to the concrete mix. The snow on the aggregates increased the water to cement ratio decreasing the strength of the concrete. It should also be noted that due to an accelerator admixture used in the concrete mix that poor B had considerable consolidation issues. The concrete began to harden before the forms could be properly vibrated to consolidate the concrete. This was not an issue in pour A as the extra water added from the snow made a much more flowable concrete. The results of the compressive strength tests can be seen in Figure 6a.

Two axial tension tests were performed to ensure that the welds being used to create the 10M steel reinforcing grids would not affect the strength of the steel. The first tension test was performed on a length of reinforcing steel with no weld and the second test was performed on a separate piece of reinforcing steel with a weld at mid-height. Figure 6b shows that the welds had no effect on the strength of the reinforcing steel and minimal effect on ductility. The panels were designed assuming a yield strength of 400 MPa for the reinforcing steel. The measured yield strength and ultimate strength of the reinforcing steel is provided in Figure 6b.



a) Concrete Compressive Strength b) Steel Reinforcing

Figure 6: Mechanical Properties of Concrete & Reinforcing Steel

4.3 Pure Flexure Tests

The first tests performed on the wall panels were under pure flexural loads. One panel from each concrete pour was tested in positive and negative flexure under quarter point loading until failure. Moment due to lateral load was determined with the results of these tests will be discussed below.

4.3.1 Panel A-PF

Panel A-PF was in good condition prior to testing. The panel had a slight downward camber with a few surface cracks noted on the tension face. These issues were most likely due to handling during demoulding. There were also light consolidation issues visible within the concrete. However, these issues were minor and were not likely to affect the performance of the wall panel in flexure. Panel A-PF was tested under steadily increasing quarter point loading until failure. As seen in Figure 7a, the ultimate moment capacity of the panel reached 10.56 kN-m under a peak lateral load of 35.6 kN which is well in exceedance of the positive design moment. At this point the steel reinforcing began to yield and hold load while still deforming. The concrete then crushed after the steel yielded and the panel was unloaded. The fact that the steel yielded before the concrete crushed shows a desirable, ductile failure mode of the wall panel.

4.3.2 Panel A-NF

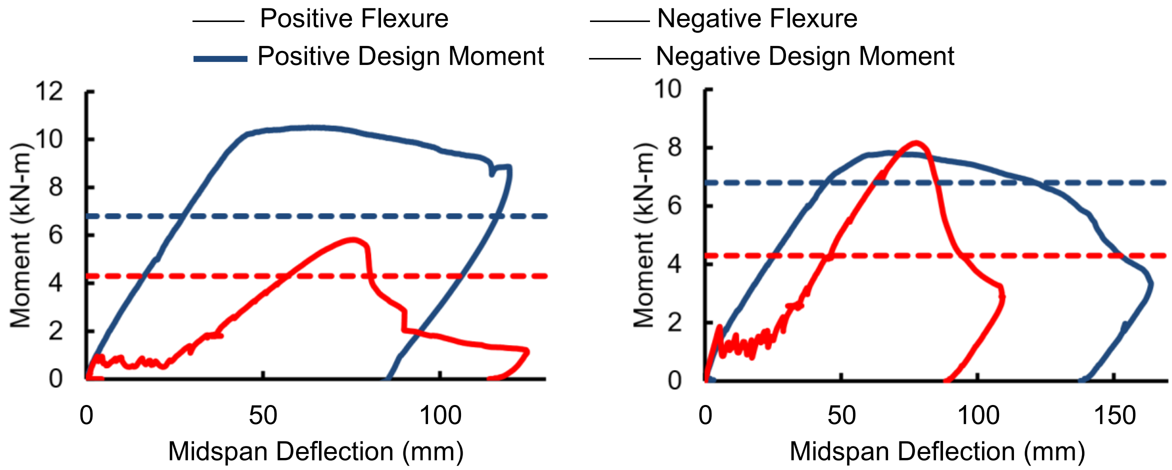
Panel A-NF was in good condition prior to testing. Small surface cracks were visible on the surface of the panel, however, these cracks were located on the compression face of the panel and would not affect the performance of the concrete in bending. The panel had a slight upward camber when placed on the testing apparatus. The surface cracks and camber are likely due to shrinkage and poor handling during demoulding. Panel A-NF was tested under steadily increasing quarter point loading until failure. As seen in Figure 7a, the ultimate moment capacity of the panel reached 5.86 kN-m under a peak lateral load of 19.73 kN which is in exceedance of the negative design moment. At this point the load dropped abruptly and the panel failed. This sudden drop in load indicates that the concrete crushed before the reinforcing steel yielded. This could be due to improper placement of the reinforcing steel or the compressive strength of pour A being lower than the design value. It should also be noted that panel deflected approximately 20mm before the moment began to increase. This was likely due to pre-camber cracks closing while the initial load was being applied.

4.3.3 Panel B-PF

Panel B-PF had considerable consolidation issues that could be seen on the tension face of the specimen. This was likely not an issue as the concrete cover allowed the steel reinforcing to develop full bond. Panel B-PF was tested under steadily increasing quarter point loading until failure. As seen in Figure 7b the ultimate moment capacity of the panel reached 7.87 kN-m under a peak lateral load of 26.56 kN. At this point the steel reinforcing began to yield and load gradually decreased until failure. Same as panel A-PF this indicates a favourable ductile failure mode.

4.3.4 Panel B-NF

Panel B-NF was in overall good condition prior to testing with slight consolidation issues noted on the compression face, however, minimal cross section was lost. Panel B-PF was tested under steadily increasing quarter point loading until failure. As seen in Figure 7b the ultimate moment capacity of the panel reached 8.17 kN-m under a peak lateral load of 27.7 kN. At this point the load dropped abruptly, and the panel continued to deform until failure. This indicates that the concrete crushed before the reinforcing steel yielded causing a brittle failure. This could have been caused by the consolidation issues on the compression face of the panel or improper placement of the reinforcing steel. Again, like A-NF, the panel deflected approximately 20 mm before the moment began to increase.



a) Pour A Panels   b) Pour B Panels

Figure 7: Panels in Pure Flexure

4.4 Combined Axial Compression and Flexure Tests

The second set of tests performed on the wall panels were under combined axial and flexural loads. One panel from each concrete pour was tested in positive and negative flexure under quarter point loading with axial load until failure. Each panel was loaded in axial compression up to approximately 10% of the panels concentric capacity and then loaded in flexure until failure. Moment due to the lateral load was determined and is considered the primary moment. Moment due to axial load, P-delta or secondary moments, were also considered. These secondary moments were caused by the midspan deflection as well as the eccentricity between the applied axial load and the centroid of the panel cross section. The results of these tests will be discussed below.

4.4.1 Panel A-A-PF

Panel A-A-PF was in good condition prior to testing with no visible cracks or initial camber present. A small portion of concrete was pried off from the corner of the panel during demoulding which would not affect the results of the test. Panel A-A-PF was first loaded in axial compression up to 90 kN. Lateral load was then gradually applied, and the panel was monitored until failure at a peak lateral load of approximately 23 kN. The primary moment due to lateral load reached a peak of 6.83 kN-m and the moment due to secondary effects reached a peak of 5.24 kN-m. As seen in Figure 8a, these peaks did not occur at the same time. The peak total moment reached approximately 11.4 kN-m at which point the load began to gradually decrease until failure. The failure of this panel was dominated by the yielding of the reinforcing steel. The negative slope in the total moment vs. deformation curve is likely due to the secondary effects.

4.4.2 Panel A-A-NF

Panel A-A-NF was in good condition with no missing concrete, consolidation issues, or pre-camber present.

Narrow cracks were observed on the tension face of the panel, likely due to improper handling during demolding and shrinkage. These cracks were insignificant and did not affect the capacity of the panel. Panel A-A-NF was first loaded in axial compression up to approximately 80 kN. Lateral load was then gradually applied, and the panel was monitored until failure at a peak lateral load of approximately 10.5 kN. Figure 8b shows that the panel had deformed approximately 7mm before the lateral load was applied. This initial deflection happened while the panel was being loaded in axial compression and was caused by the eccentric load application. The primary moment due to lateral load reached a peak of 3.10 kN-m and the moment due to secondary effects reached a peak of 6.31 kN-m. The peak total moment reached approximately 9.12 kN-m at which point the load decreased abruptly and the panel failed. This indicates that the concrete crushed before the reinforcing steel yielded resulting in a brittle failure.

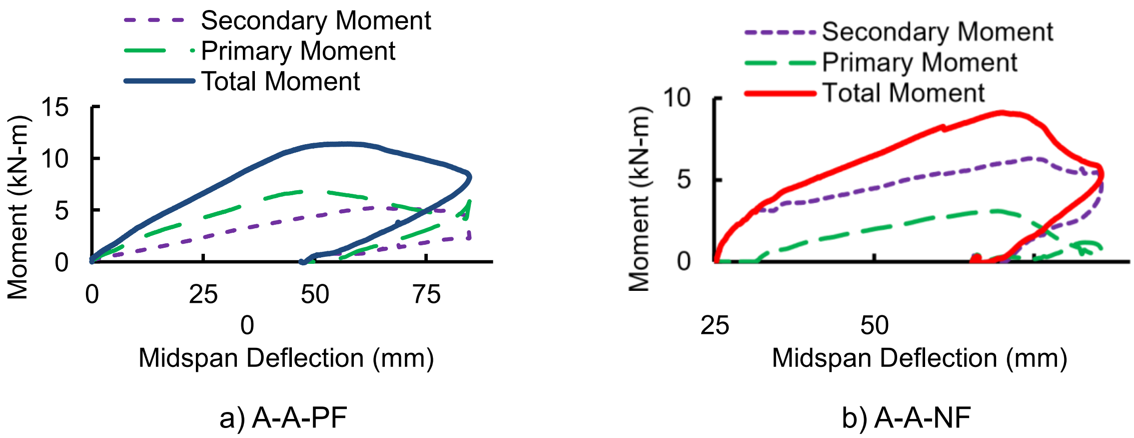


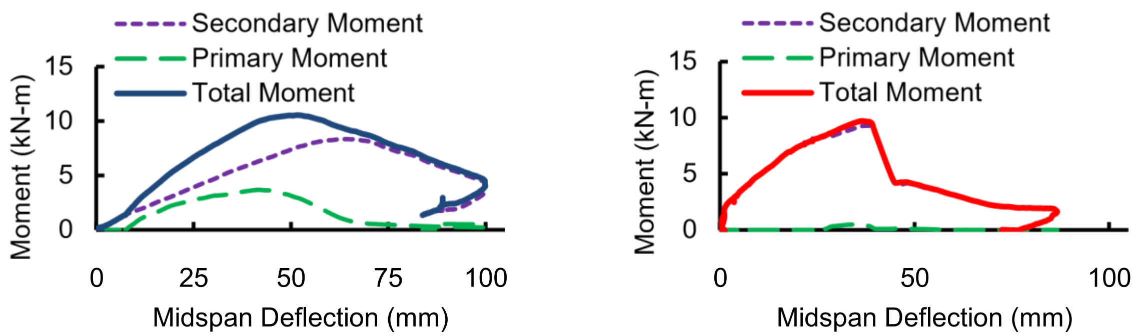
Figure 8: Primary, Secondary, and Total Moments of Panels A-A-PF & A-A-NF

4.4.3 Panel B-A-PF

Considerable consolidation issues were present throughout this wall panel. As this panel was tested in positive flexure, the voids caused by the poor consolidation were on the tension face of the specimen. Due to the concrete cover, this would not affect the development of the reinforcing steel and the overall performance should be minimally affected. An initial downward camber measuring approximately 20 mm was also present in this wall panel. Panel B-A-PF was first loaded in axial compression up to approximately 150 kN. Lateral load was then gradually applied, and the panel was monitored until failure at a peak lateral load of approximately 12.5 kN. Figure 9a shows that the panel had deformed approximately 7.5mm before the lateral load was applied. This initial deflection happened while the panel was being loaded in axial compression and was caused by a combination of the eccentric load application and the initial camber in the panel. The primary moment due to lateral load reached a peak of 3.7 kN-m and the moment due to secondary effects reached a peak of 8.34 kN-m. The peak total moment reached approximately 10.6 kNm at which point the lateral load quickly fell off, however, the axial load held and began to gradually decrease until the panel failed. The failure of this panel was dominated by the yielding of the reinforcing steel. The negative slope in the total moment vs. deformation curve is likely due to the secondary effects.

4.4.4 Panel B-A-NF

Severe consolidation issues were present on the compression side of this panel and light tension cracks were present on the tension face. No camber was noted in this panel prior to testing. Panel B-A-NF was first loaded in axial compression up to approximately 145 kN. Lateral load was then gradually applied, and the panel was monitored until failure with a peak lateral load of only 1.7 kN. As seen in Figure 9b, the midspan deflection had already reached 22 mm at the time of lateral load application. This initial deflection occurred while the panel was being loaded in axial compression and was caused by a combination of the eccentric axial loading and the severe consolidation issues on the compression face of the panel. The voids created by the poorly consolidated concrete were closed quickly during the axial loading and accelerated the midspan deflection. This is another reason why the panel was able to take almost no flexural load. The primary moment due to lateral load only reached a peak of 0.5 kN-m. The moment due to secondary effects, which dominated the behaviour of this panel, reached a peak of 9.3 kN-m. The peak total moment reached approximately 9.7 kN-m at which point the concrete crushed and the lateral load dropped off. The load was then picked up by the reinforcing steel and the panel continued to deform until the load was removed and the test was terminated. The failure of this panel occurred when the concrete crushed prior to the reinforcing steel yielding causing a brittle failure. This is likely due to the severe consolidation issues in the compression face and/or improper placement of the reinforcing steel.

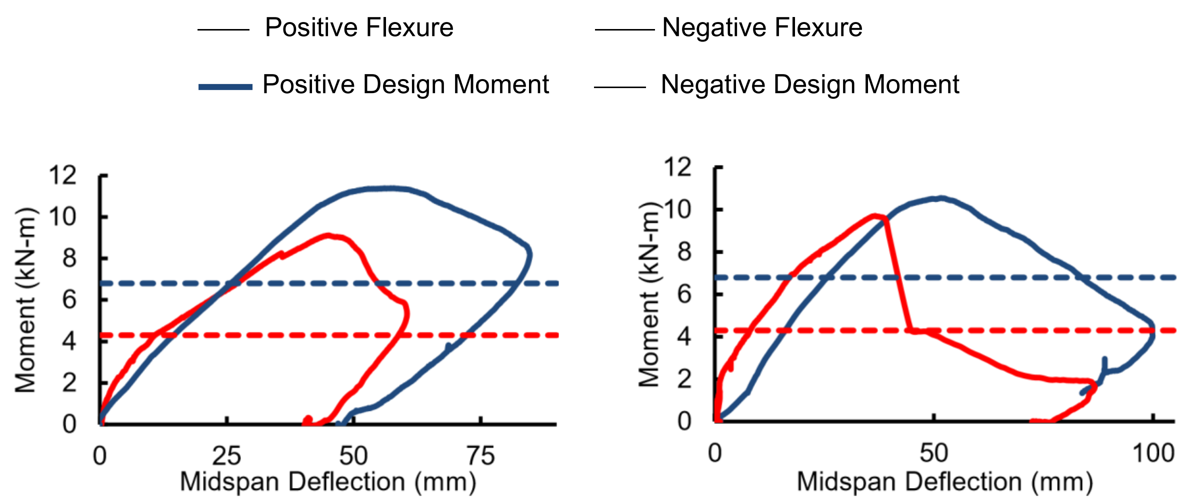


a) B-A-PF

b) B-A-NF

Figure 9: Primary, Secondary, and Total Moments of Panels B-A-PF & B-A-NF

A comparison of the total moment between pour A and Pour B for axial compression in both negative and positive flexure can be seen in Figure 10. It can be seen that even though panels A-A-NF and B-A-NF had brittle failure modes, both panels reached their design moment capacity.



b) Pour B Panels

a) Pour A Panels

Figure 10: Panels in Combined Axial Compression and Flexure

5 CONCLUSIONS

The testing program completed on the eight waffle panels considered in this paper show that EPS concrete used in conjunction with a waffle panel design can be a viable option for wall panels when resisting category 5 hurricane wind loads. All of the panels tested in negative bending failed in a brittle fashion with the concrete crushing before the steel yielded however, all of the panels reached a total moment resistance well above their design moment. Issues in concrete pour A that lowered the compressive strength below the design strength and consolidation issues in pour B that created voids in the compression face when testing the panels in negative flexure likely caused the panels tested in negative flexure to have a brittle failure. Improper placement of the reinforcing steel could have also led to the issues with the panels in negative flexure. With consistent material properties and reliable placement of reinforcement that can be provided in precast concrete plants this paper demonstrates that concrete lightweight EPS concrete waffle panels could be an attractive building material for modular construction.

6 ACKNOWLEDGEMENTS

The authors would like to thank the New Brunswick Innovation Foundation (NBIF) for their financial contributions to this research. Assistance in the laboratories from Andrew Sutherland and Chris Forbes was greatly appreciated.

7 REFERENCES

Aaleti, S. R., Sritharan, S., Bierwagen, D., & Wipf, T. J. 2011. Structural Behavior of Waffle Bridge Deck Panels and Connections of Precast Ultra-High-Performance Concrete. *Journal of the Transportation Research Board*, 82-92.

Abdul-Wahab, M. H., & Khalil, M. H. 2000. Rigidity and Strength of Orthotropic Reinforced Concrete Waffle Slabs. *Journal of Structural Engineering*, 219-227.

Alqahtani, F. K., Ghataora, G., Khan, M. I., & Dirar, S. 2017. Novel lightweight concrete containing manufactured plastic aggregate. *Construction and Building Materials*, 386-397.

ASTM. 2022. Standard Test Methods of Conducting Strength Tests of Panels for Building Construction. Standard ASTM E72-22, West Conshohocken, PA.

Cook, D. 1973. Expanded polystyrene beads as lightweight aggregate for concrete. *Precast Concrete*, 691693.

CSA. 2014. Design of concrete structures. Standard CAN/CSA-A23.3-14, Canadian Standards Association, Toronto, Ont.

Einea, A., Salmon, D. C., Fogarasi, G. J., Culp, T. D., & Tadros, M. K. 1991. State-of-the-Art of Precast Concrete Sandwich Panels. *PCI Journal*, 78-98.

Freedman, S. S. 1999. Loadbearing architectural precast concrete wall panels. *Precast Concrete Institute*, 92-115.

Jiang, J., Lu, Z., Niu, Y., Li, J., & Zhang, Y. 2016. Study on the preparation and properties of high-porosity foamed concretes based on ordinary Portland cement. *Materials & Design*, pp. 949-959.

Joseph, J. D. 2020. Precast Concrete Sandwich Panels for Mass Housing Systems: Plan and Design Strength Requirements. *The Institution of Engineers (India)*, 359-368.

Nasser, G. D., Tadros, M., Sevenker, A., & Nasser, D. 2015. The legacy and future of an American icon: The precast, prestressed concrete double tee. *Precast Concrete Institute*, 49-67.

Nikbin, I. M., & Golshekan, M. 2018. The Effect of Expanded Polystyrene Synthetic Particles on the Fracture Paramaters, Brittleness and Mechanical Properties of Concrete. *Theoretical and Applied Fracture Mechanics*, 160-172.

Seifi, P., Henry, R. S., & Ingham, J. M. 2019. In-plane cyclic testing of precast concrete wall panels with grouted metal duct base connections. *Engineering Structures*, 85-98.

Tomek, R. 2017. Advantages of precast concrete in highway infrastructure construction. Creative Construction Conference pp. 19-22.

Xiong, C., Chu, M., Liu, J., & Sun, Z. 2018. Shear behavior of precast concrete wall structure based on twoway hollow-core precast panels. *Engineering Structures*, 74-89.