Real time monitoring of concrete – lattice-girder flat slabs during construction

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Abstract

This paper reports the instrumentation, monitoring and some preliminary results from a real time monitoring scheme of a hybrid concrete lattice girder flat slab floor system. One of the key benefits of this monitoring project is the improved understanding of insitu structural behaviour. Sensors were embedded in both the precast and insitu components of the hybrid concrete floor system and are used to monitor various aspects of the behaviour of the floor during the manufacture, construction and operational phase of the building. Concrete strains and temperatures were measured, while environmental conditions were also monitored to assess their effects on the floor structure. The information from the real time monitoring offers the opportunity to compare actual and predicted behaviour using structural codes, such as Eurocodes. Some of the results on the early-age behaviour of the concrete floor slab during the construction phase (strain, thermal cracking, restraint factors) are described in this paper.

Keywords chosen from ICE Publishing list

Buildings, structures & design; Concrete structures; Field testing & monitoring

List of notation (examples below)

Cv	is the coefficient of variation
E_{cm}	is the mean elastic modulus of concrete
ER	is electrical resistance (strain gauge)
f_{ck}	is the characteristic compressive strength of cylinder at 28 Days
f_{ctm}	is the mean tensile strength of concrete
FE	is finite element
GGBS	is ground granulated blast-furnace slag
GL	is gridline
HBB	is the Human Biology Building
R	is the restraint factor
RH	is relative humidity
SHM	is structural health monitoring
TDE	is time dependent effects
VW	is vibrating wire (strain gauge)
α_c	is the coefficient of thermal expansion of concrete
α_{vw}	is the coefficient of thermal expansion of the vibrating wire gauge
α_r	is the restrained coefficient of thermal expansion of concrete
$\Delta\varepsilon$	is the change in strain
ΔT	is the change in temperature
ε_{ctu}	is the tensile strain capacity of concrete
ε_r	is the restrained strain

1. Introduction

This paper describes the implementation of a real time structural health monitoring (SHM) scheme on a new building (Human Biology Building) under construction at the National University of Ireland Galway (NUI Galway). This project is one of a number of projects which forms part of a measurement framework strategy developed at NUI Galway to continuously monitor the structural and environmental performance of buildings during construction and operation (Goggins *et al.*, 2014; Hajdukiewicz *et al.*, 2015; Newell *et al*., 2016). The general SHM methodology for these projects is described in more detail by Newell *et al*. (2016). In previous projects constructed on the NUI Galway campus, the structural behaviour of a void form flat slab was monitored in the Engineering Building (EB) (Byrne *et al.*, 2012) and the environmental and thermal performance of precast components was monitored in the Institute for Lifecourse and Society Building (ILAS) (Hajdukiewicz *et al.*, 2015). The Engineering Building, completed in 2011, also acts as a 'Living Laboratory' for engineering students (Goggins et al 2012). Although the HBB is under construction at the time of writing, some preliminary results from the project are presented in this paper, primarily focussing on the early-age behaviour of a hybrid concrete precast concrete lattice girder flat slab during the construction phase. This project is part of the Built2Spec European project in which the structural and environmental performance of buildings are continuously monitored using smart sensor-embedded construction elements (smart building components).

Although the majority of sensors and monitoring equipment are predominantly used on civil infrastructure projects, the use of instrumentation for the design, construction and management of buildings has increased in recent decades. The trend towards increased service life and durability of structures means that monitoring systems which permit the condition of the structure to be measured and which detect the possible onset of damage and/or deterioration of the structure will become increasingly important. A SHM system, if correctly designed and implemented, can be used to assess the current condition of the structure, residual life prediction and detection of deterioration of a structural component (Buenfeld *et al.*, 2008). One of the key benefits of insitu instrumentation is the improved understanding of insitu structural behaviour. The majority of design codes have been developed following research conducted in engineering laboratories, where it can be very difficult to replicate the behaviour of real structures insitu. Consequently, testing and analysis is typically conducted on small-scale specimens which only partially represent the overall structure. Real time monitoring can be used to provide rich information on how real structures behave when subject to actual structural and environmental loads (ISIS Canada, 2005).

A variety of sensors were embedded in the hybrid concrete floor structure of the building and are used to monitor various aspects of the behaviour of the floor during the manufacture, construction and operational phase of the building.
The overall objectives of this research project are:

- compare actual and predicted behaviour of the floor.
- analyse the long term behaviour of the floor.
- investigate construction and design parameters for potential optimisation of hybrid concrete floor systems.
- develop and calibrate numerical models that predict the performance of the hybrid concrete floor system.

Figure 1. Human Biology Building (under construction)

2. Human Biology Building

The Human Biology building (HBB) is a four storey building over basement and roof level plant enclosure with a gross floor area of 8200m^2 (Figure 1). This building will house the existing university disciplines of Anatomy, Physiology & Pharmacology and Therapeutics. The building has been designed as a teaching and research facility with accommodation including teaching and research laboratories, offices, anatomy and mortuary facilities, meeting rooms, lecture theatres and other ancillary areas. Construction commenced in January 2015 and the building was handed over to the client in March 2017. It is anticipated that the building will achieve an A rating under the Commercial Energy Rating marking scheme and a BREEAM Excellent rating. The HBB is primarily constructed using precast concrete elements, including the building frame, twinwall system, hybrid concrete lattice girder slabs and hollowcore slabs, which were designed, manufactured and installed by Oran Pre-Cast Ltd. in Galway, Ireland.

2.1. Hybrid concrete lattice girder flat slab

A two-way spanning hybrid precast concrete lattice girder flat slab system is used for the floor structure. The majority of the floor plate is 400mm thick and consists of a 65mm thick precast lattice girder plank (Figure 2) and 335mm insitu concrete topping. The lattice girder truss, which protrudes from the plank, provides stiffness in the temporary state and increases composite action with the insitu structural concrete topping. The bottom layer of reinforcement is contained within the precast concrete plank and the top layer of reinforcement is placed on site, together with a layer of stitching reinforcement across the joint between slabs, prior to pouring the insitu concrete topping. The precast planks are temporarily propped until the structural concrete topping has reached the required compressive strength.

Figure 2. Precast Concrete Lattice Girder Plank (with embedded sensors)

A reinforced concrete twin wall system is the primary lateral resisting system in the building, as well as transferring gravity loads to the ground. The twin wall system consists of two plates of 65mm thick concrete connected by means of cast-in lattice girders to form a core between the plates. The cavity between the plates is filled with concrete on site after the panel has been erected to complete the composite wall. Precast reinforced concrete columns and downstand beams are also employed in the building.

Flat slabs are one of the most widely used forms of floor construction providing minimum structural depths, fast construction and uninterrupted service zones. Transportation and site handling limitations generally dictate the allowable sizes of the precast planks. The use of steel moulds results in a high-quality finish, which can be left exposed if required. The quality of the factory produced soffits also provides the opportunity to take advantage of the thermal mass properties of the concrete slab by exposing them. The main advantages of this floor system to the contractor are in terms of programme, reduction in steel fixing and formwork requirements on site.

3. Instrumentation

The SHM strategy implemented in the HBB employed sensors embedded in the floor structure to monitor the environmental and structural behaviour of the hybrid concrete lattice girder flat slab. Two zones in the second floor of the HBB were selected for instrumentation using a combination of vibrating-wire (VW) strain gauges, electrical resistance (ER) strain gauges and thermistors (Figure 3 and 4).

Thermistors **VW gauges** **ER gauges**

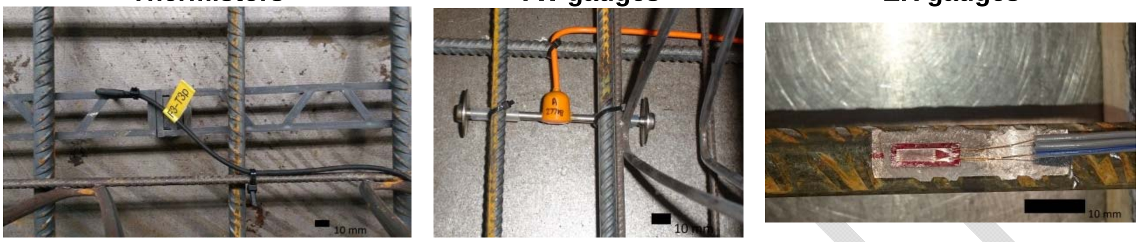


Figure 3. Sensors embedded in the building's structure

A total of 112 embedment VW strain gauges, manufactured by Gage Technique (Type TES/5.5/T), were installed in both the precast plank prior to manufacture and in the insitu concrete topping prior to pouring. The VW gauges measure both temperature and longitudinal strain in the concrete. These type of gauges were initially developed by the Transport and Road Research laboratory (TRRL) in the UK and are used extensively in bridge sections, tunnel linings and dam projects. They have a range of greater than 300 microstrain and resolution better than 1 microstrain. The temperature can be measured between -20°C and +80°C. These VW gauges are very robust and their stability makes them suitable for monitoring time dependent effects in concrete such as creep and shrinkage.

The strain gauge operates on the principle that a tensioned wire, when plucked, vibrates at a frequency that is proportional to the strain in the wire. The gauge is constructed so that a wire is held in tension between two end flanges. Loading of the concrete structure changes the distance between the two flanges and results in a change in the tension of the wire. An electromagnet is used to pluck the wire and measure the frequency of vibration. Strain is then calculated by applying calibration factors to the frequency measurement (Gage Technique, 2016). In order to interpret the data from the VW strain gauges, it is very important that strain readings are corrected for temperature effects. The internal temperature of the concrete in the floor, as well as causing expansion and contraction of the concrete, will also change the temperature of the VW gauge which will affect the readings from the VW strain gauges. The coefficient of thermal expansion of the VW strain gauges is 11$\mu\varepsilon$/°C and for this project it was assumed that the coefficient of thermal expansion of the concrete was 9$\mu\varepsilon$/°C, as limestone aggregates were used for the concrete in the precast plank and insitu topping (Bamforth *et al.*, 2008). Therefore, in order to measure the strain of the concrete due to load effects only, the difference in thermal expansion between the gauge and host material must be accounted for ($2\mu\varepsilon/°C$, in this instance).

The VW gauges were positioned along a number of orthogonal grids in the floor structure at over 30 designated locations so that two-way spanning behaviour of the floor structure could be monitored. At most locations, four VW gauges were positioned through the depth of the slab (1 in the precast plank and 3 in the insitu topping) so that the strain and temperature profile through the slab could be measured (Figure 4). The location of the embedded sensors in one zone of the second floor is shown in Figure 5.

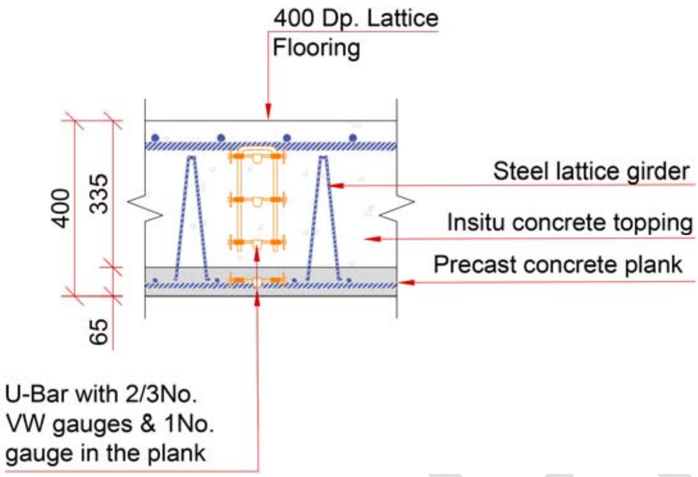


Figure 4. Typical section showing locations of vibrating wire gauge installed in the flat slab system.

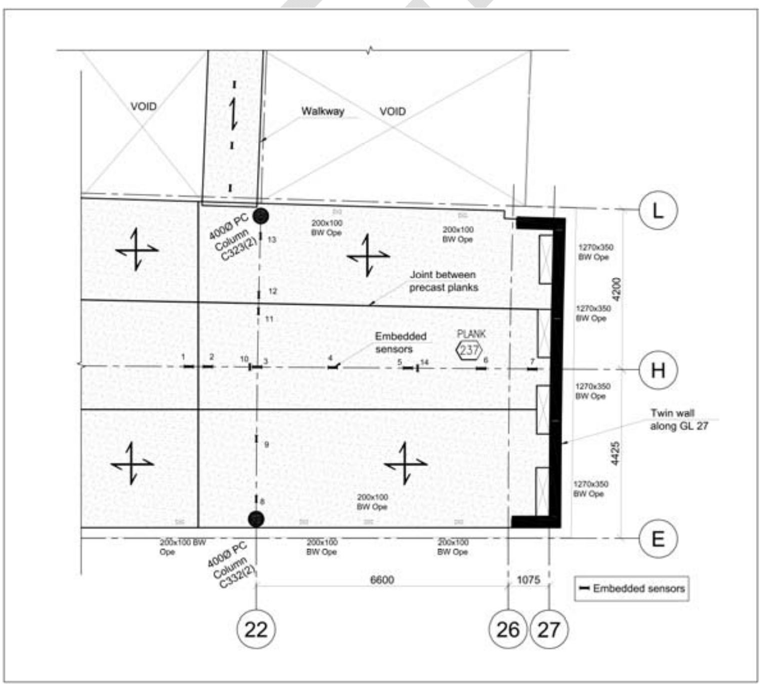


Figure 5. Part of second floor slab of HBB with insitu instrumentation

3.1. *Material testing*

To accurately predict the behaviour of concrete elements, it is critical that the properties of the concrete are determined as they will change over time depending on the environment and loading (Neville, 1995). A comprehensive material testing programme was undertaken to measure the properties of the precast and insitu concrete used in the floor structure. The information from the material testing is used to interpret the data from the instrumentation and can also be used for modelling of the behaviour of the floor structure. Concrete cylinders, cubes and prisms specimens were made and cured in water and in air (to match the environmental conditions of the floor structure).

The precast planks were manufactured using a C40/50 concrete mix (BSI, 2013) with a CEM II A-V cement (370kg/m^3 typically or 425kg/m^3 of self-compacting concrete used). The mix design for the insitu concrete topping was C30/37 with 230kg/m^3 of CEM I cement and 100kg/m^3 of GGBS (i.e. 30%).

3.2. *Environmental conditions*

The environmental conditions around the floor structure will impact on the behaviour of the concrete slab, particularly at early stages of curing. Air temperature and relative humidity in the vicinity of the instrumented slab were measured using the weather data from the NUIG weather station (located approximately 1km from the site) (IRUSE, 2016). When building was enclosed a sensor was positioned inside the building to record the air temperature and relative humidity.

4. Results

As mentioned previously, the HBB is still under construction but all of the insitu instrumentation will continue to monitor the behaviour of the floor structure during the operational phase of the building. This will allow long-term effects of creep and shrinkage of concrete elements to be further investigated. This paper presents some of the results from the sensors focussing on the early-age behaviour of the hybrid concrete lattice girder flat slab during the construction phase.

4.1. *Temperature rise and temperature differentials*

Predicting the temperature rise and temperature differentials are key parameters when determining the early-age behaviour of concrete elements and the potential for thermal cracking. However, at design stage, this can sometimes be difficult as there are numerous factors which affect the early-age behaviour of concrete. Guidance in relation to predicting the early–age thermal behaviour of concrete sections is provided in the CIRIA Report C660 by Bamforth (2007). However, the report recommends thermal modelling for reliable predictions which take account of the formwork and exposure conditions. Insitu instrumentation can also be used to determine parameters, such as peak temperature and temperature differentials in concrete pours, to improve the accuracy of prediction models for early-age thermal cracking.

Nine VW strain gauges were embedded in one of the 65mm thick precast planks (Plank 237, Figure 5) prior to manufacture and data was recorded immediately after the plank was cast. The concrete temperature for the nine VW gauges embedded in the plank and the air temperature (dashed line) are shown in Figure 6 for the 7 days after the plank is manufactured. The peak in the concrete temperature occurs approximately 10 hours after casting, which is typical of the heat generation phase during cement hydration (Mindess *et al*., 2002) and contrasts with the falling ambient air temperature during the night. The max recorded difference between the air temperature and the concrete temperature is approximately 5.5°C. The peak temperature for the three gauges (No. 2, 7 and 11) located close to the perimeter (less than 300mm) are slightly less as the concrete adjacent to the external surface cools at a faster rate. Within 24 hours after manufacture, the concrete temperature in the plank is relatively uniform for all nine VW gauges and they correlate with the ambient air temperature because the plank is relatively thin (65mm).

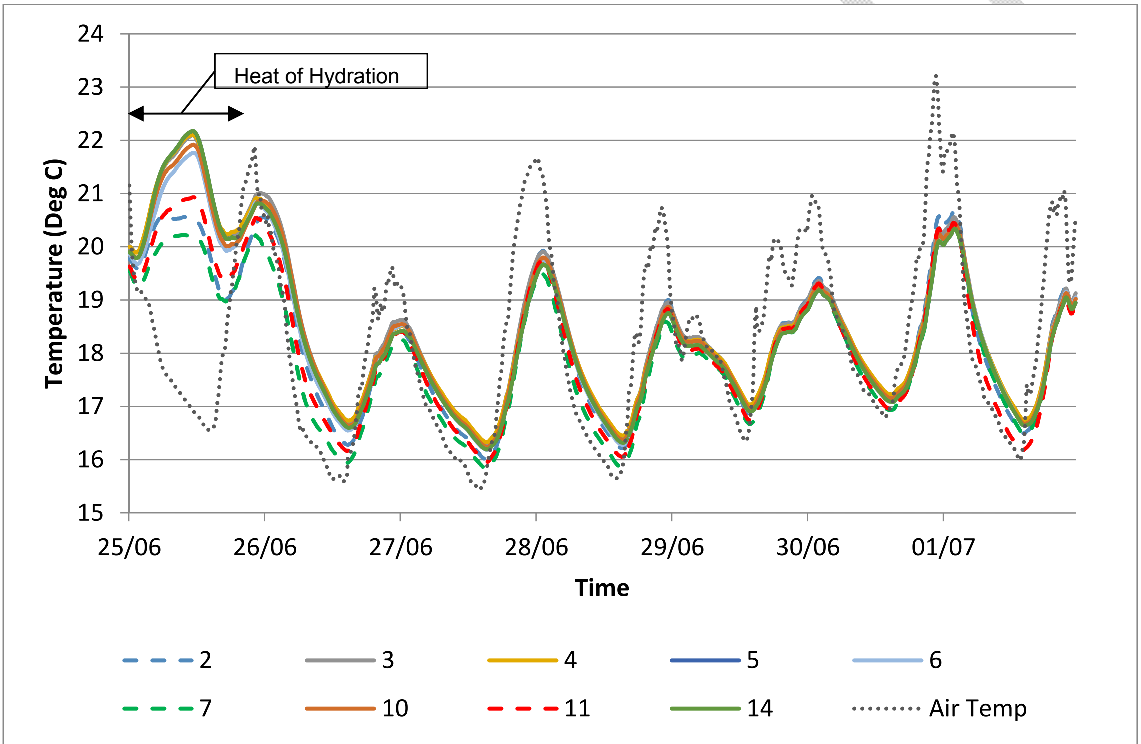


Figure 6. Concrete temperature in a 65mm thick precast plank (7 days after manufacture)

The concrete temperatures in the insitu topping at one location in which three VW gauges are embedded to measure strain and temperature through the insitu topping (top, middle and bottom) and precast plank are shown in Figure 7. The peak temperatures recorded in the insitu topping are more significant than the precast plank because of the thickness of the concrete (335mm thick) and the insulating effect of the precast biscuit which means that only one surface of the insitu topping is directly exposed to air. Similar to the precast plank, the peak in concrete temperature occurs approximately 10 hours after casting and at this point, the maximum temperature differential between the air and concrete temperature is approximately 13°C. It takes approximately 7 days for the heat generated from the hydration process to dissipate. The temperature in the insitu topping exceeds the ambient air temperature for approximately 7 days after the pour. The effect of the diurnal temperature changes are also clearly visible in the measured temperature in the concrete floor. The peak temperature is recorded for the VW gauge in the middle of the insitu topping as the internal section of the slab will be slowest to cool down. The effect of the heat of hydration from the insitu topping on the concrete in the precast plank can be noted almost immediately after pouring and results in a peak increase in temperature of 9°C.

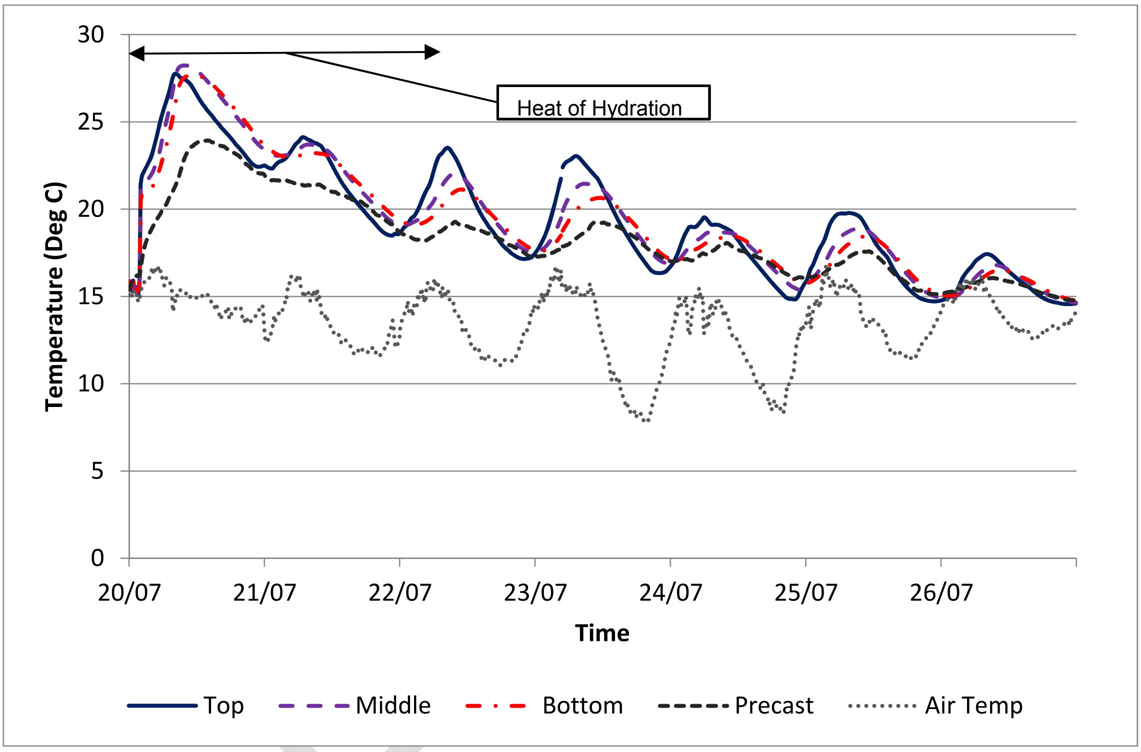


Figure 7. Concrete temperature in the 335mm insitu precast plank (7 days after pour)

As expected, the peak temperature in the concrete during the initial hydration phase was recorded in the middle section of the insitu topping and the lowest temperature is recorded in the top section due to heat loss to the surrounding air. The concrete in the bottom section of the insitu is partially insulated by the precast plank but the heat transfer to the plank is noted as the temperature of the precast plank increases during the hydration phase. After the initial hydration phase, the concrete temperature in the slab is primarily in response to the diurnal temperature changes. The time lag between the concrete temperatures in the insitu topping and the ambient air temperature are shown in Figure 7. The time lag increases from top to bottom through the 335mm thick insitu topping as the top of the slab is exposed and is subject to radiant heating from the sun. In contrast, the time lag recorded between the ambient air temperature and the concrete in the precast plank (Figure 6) after manufacture is much smaller because it is only 65mm thick and consequently has a much smaller thermal mass in comparison with the insitu topping.

One of the main concerns with large volume pours is the heat generated as the cement hydrates and the potential thermal stresses generated by restraint to thermal movement. In terms of minimising cracks, the general rule of thumb used by designers is to limit temperature differentials to 20°C, although this figure can be revised depending on the type of aggregate used in the concrete mix. For the insitu topping in this project, the temperature differential through the slab is small (Figure 7) because of the relatively thin section which allows the concrete to cool comparatively uniformly as the heat is readily lost to the environment. The temperature differential recorded by the VW gauges in top, middle and bottom of the insitu topping did not exceed 3°C and the maximum differential occurred during the cooling phase. Using CIRIA Report C660 (Bamforth, 2007), the predicted maximum temperature differential in the insitu topping is 13°C, but this figure is highly dependent on the thermal diffusivity of concrete, surface conditions and the environmental conditions (wind and solar gain). The predicted peak temperature differential for the insitu topping using CIRIA Report C660 (Bamforth, 2007) is approximately 19°C, but other factors which can affect the peak temperature are the variation between cements, placing temperature and actual thermal conductivity of the precast plank. The measured peak temperature of 13°C equates to a temperature rise of 4°C/100kg per cubic metre of concrete. In this project, the mix design for the insitu concrete topping was C30/37 with 230kg/m^3 of CEM I cement and 100kg/m^3 of GGBS (i.e. 30%). Fitzgibbon (1976) estimated that the peak temperature rise under adiabatic conditions is 12°C/100kg per cubic metre of concrete, regardless of the type of cement used. This figure can be considered as an upper bound, but does not take into account the reduced heat of hydration because of the 30% GGBS used as a cement replacement in the concrete mix.

4.2. Early-age thermal cracking

During the early stages of curing, there is potential for cracking to occur if the tensile strain capacity of the concrete is exceeded. Some research has shown that microcracks can form even if 50% of the tensile strength of the concrete is exceeded (FIB, 2001). The coefficient of thermal expansion of concrete (α_c) will determine the magnitude of thermal strain associated with a particular temperature change. Values for concrete vary from 8-13$\mu\varepsilon$/°C depending primarily on the aggregate used. EN 1992-1-1 (BSI, 2004) recommends a value of 10$\mu\varepsilon$/°C for normal weight concretes, if measured values are not available. However, a design value of 9$\mu\varepsilon$/°C may be used for limestone aggregates (Bamforth *et al*., 2008), which were used for the concrete on the HBB project. Using the measured peak recorded temperature of 13°C, this equates to thermal strains of approximately 117$\mu\varepsilon$ in the insitu topping. Observations have shown that early-age cracking is most likely to occur within three to six days (Alexander, 2006). The tensile strain capacity of concrete, ε_{ctu}, is the maximum strain that the concrete can withstand without the formation of a continuous crack. Tensile strain capacity is not dealt with in EN 1992-1-1, but can be derived from values of the tensile strength and elastic modulus of the concrete provided in EN 1992-1-1. Tensile strain capacity under short-term loading can be approximated as the ratio of the mean tensile strength of concrete f_{ctm} to its mean elastic modulus E_{cm} and this has been shown to represent lower bound values (Bamforth, 2007).

$$\varepsilon ctu = f_{ctm} / E_{cm}$$

1.

The values derived for tensile strain capacity are increased by 23% to take account of the relaxation of stress due to creep and reduction in tensile strength under a sustained load (Bamforth, 2007). The aggregate type is of particular significance to the tensile strain capacity as aggregate comprises about 70% of the concrete volume.

The strain profile in the insitu topping at one location in the slab for the first seven days is compared in Figure 8 with the theoretical tensile strain capacity given in Equation 1. The graph indicates the measured strain is close to or exceeds the theoretical strain capacity during the first 3 days after the pour. No cracking was observed in the location of the slab in which strain gauges were embedded and the actual tensile strain capacity of the insitu topping is at least 20-30% greater than the theoretical tensile strain capacity based on material testing conducted on concrete samples at 3 Days and 7 Days, which were air cured to match the environmental conditions on site. This illustrates the difficulty of predicting early-age cracking without accurate knowledge of the environment or material properties. Similar research based on the embedded sensors in the void form flat slab in the Engineering Building (Byrne *et al.*, 2012) also noted locations where the measured strain exceeded the theoretical strain capacity but that no cracking was observed on site.

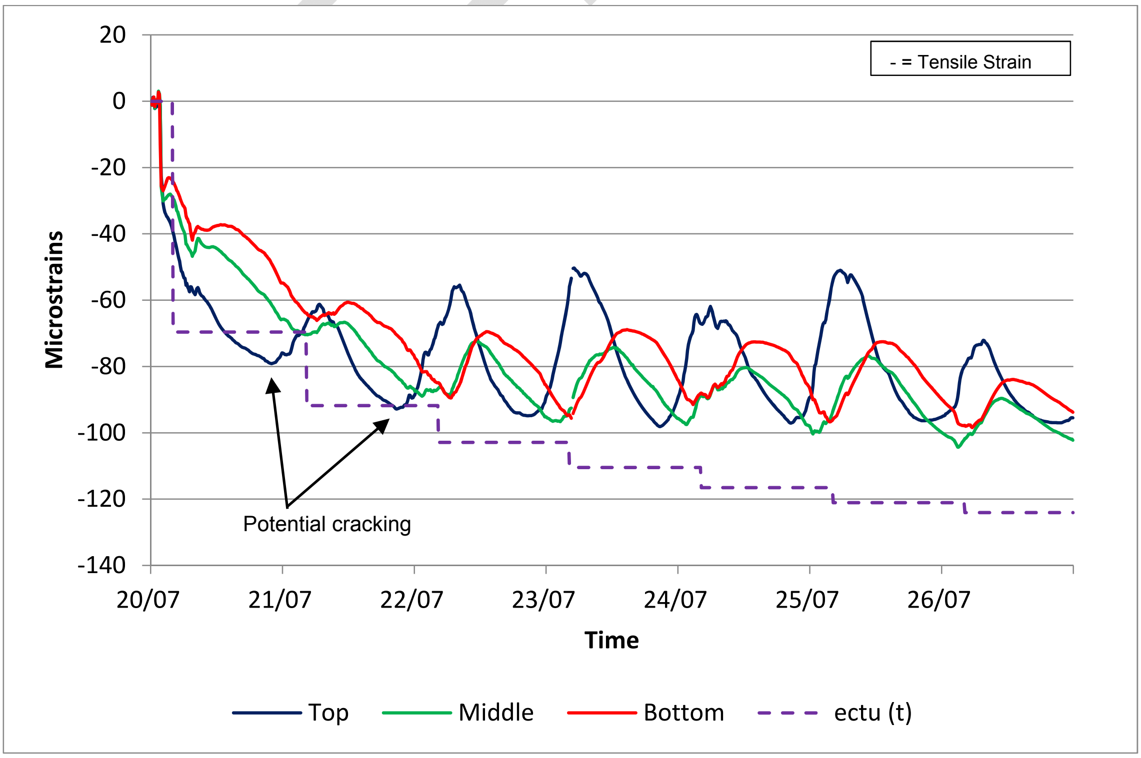


Figure 8. Comparison of measured strain and theoretical strain capacity (7 Days)

4.3. *Deriving restraint factors*

Restraint to movement can occur both externally and internally. External restraint can occur due to continuous edge restraint, end restraint and intermittent restraint. Internal restraint is where one part of a concrete pour expands or contracts relative to another part of the same section. In general, internal restraint will typically dominate for 'thick' concrete sections and external restraint will dominate for thinner sections. In this project, the hybrid concrete slab was subject to intermittent external restraint from the supporting columns and walls and internal restraint should not be critical as the insitu topping was only 335mm thick.

Early-age thermal cracking will occur if the restrained strain, ε_r, exceeds the tensile strain capacity of the concrete, ε_{ctu}. In order to estimate a value for the restrained strain, one must determine a restraint factor (R) which represents the amount of restraint to lateral and shrinkage movement which is actually provided to the structural element under consideration (R=0.0 unrestrained; R = 1.0 full restraint). Various publications provide guidance on values to be adopted for different restraint conditions, although there is little guidance with respect to suspended slabs. Information for determining restraint factors are given in Annex L of EN 1992-3 (BSI, 2006) for common construction configurations. It should be noted that the values in EN 1992-3 include a modification factor of 0.5 for creep. When calculating the risk of early-age thermal cracking, the restraint factor to use at design stage can be difficult to determine, particularly if there is the potential for internal and external restraint. Choosing an incorrect restraint factor can result in uneconomical over-design, or under-design leading to unacceptable cracking.

Insitu instrumentation can be very informative, on large projects with similar pours, to determine restraint factors. In this project, the embedded vibrating wire strain gauges were used to estimate the restraint factors for the 335mm thick insitu topping which was poured on top of the precast concrete lattice girder plank. The restraint factor is determined by comparing the observed strain in the element and the free strain that would have occurred with no restraint. The restraint factor (R) is determined as follows (Bamforth, 2007);

$$R = \frac{(\alpha_c - \alpha_r)}{\alpha_c}$$

2.

where α_c is the coefficient of thermal expansion of unrestrained concrete (derived using a Hot Box test in which in which the temperature and strain change are measured for an insulated unrestrained specimen as suggested in CIRIA Report C660) and α_r is the insitu restrained coefficient of expansion strain which is measured during the cooling phase after the concrete has reached its peak temperature after pouring. It was assumed that the free strain (α_c) or coefficient of thermal expansion of the unrestrained insitu concrete was 9με/°C (Bamforth *et al*., 2008). The insitu strain-temperature curves were plotted for the strain gauges embedded in the insitu topping and insitu restrained coefficient of expansion (α_r) is determined by measuring the slope of the strain-temperature curve during the cooling phase of the concrete. The strain-temperature curve for three VW strain gauges in the top, middle and bottom at one location in the insitu topping is shown in Figure 9. From the chart, it is observed that the restrained coefficient of thermal expansion varies between 5.7-7.6με/°C. Using equation 2, this equates to a restraint factor of 0.16-0.37, which would be considered low-moderate restraint.

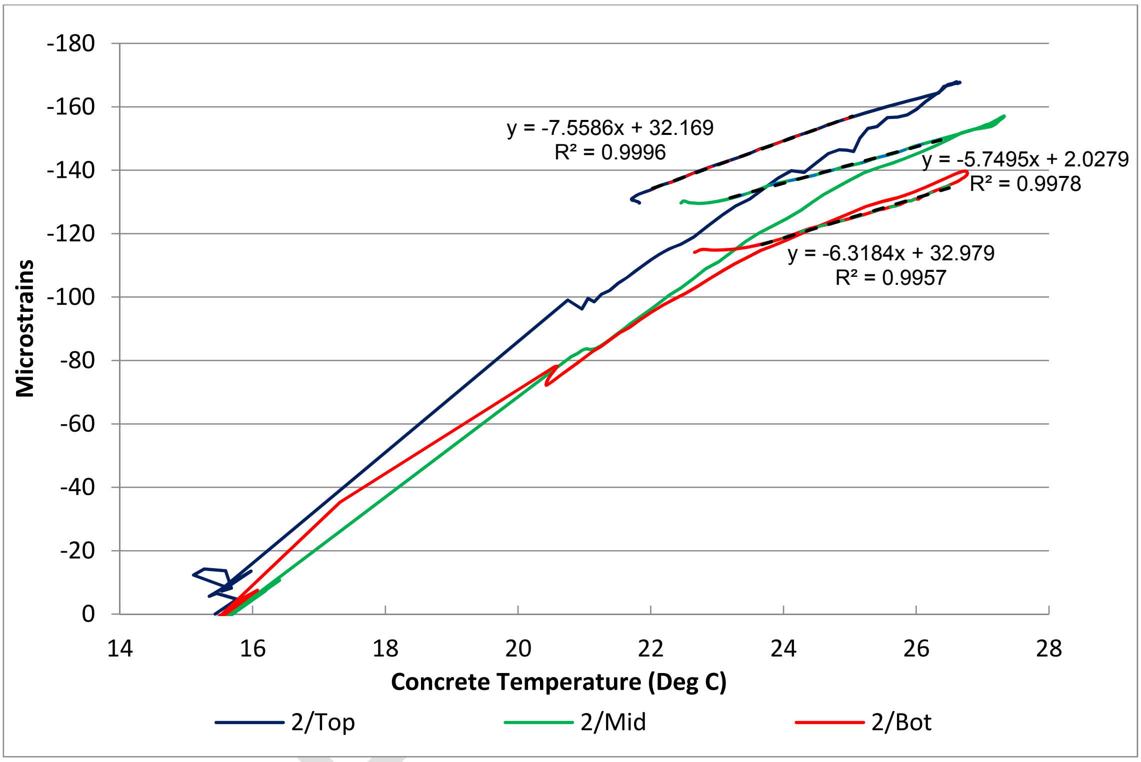


Figure 9. Insitu strain-temperature curve for insitu topping in hybrid concrete slab

The average measured restraint factor for the 335mm insitu topping was 0.27 with a coefficient of variation (Cv) of 36%. In general, the restraint factors were generally higher for the gauges in the middle of the slab (average R = 0.37, Cv = 19%) in comparison with the gauges located in the top and bottom of the insitu topping (average R = 0.23, Cv = 32%). There was no noticeable increase in the measured restraint for the gauges located close to the supporting columns or walls, which is expected as the reinforced concrete core walls in this project were constructed using the twin wall system and the insitu core of the walls were typically poured with the insitu topping so that the walls had limited stiffness during the early-age stages of the insitu topping. Using restraint factors determined using embedded instrumentation and comparison with assumed (design) values may highlight potential changes to the design that can result in more economical and cost-effective design, particularly with respect to early-age thermal cracking and resulting crack widths.

4.4. *Construction stage behaviour of hybrid concrete flat slab*

A hybrid concrete lattice girder flat slab structure was used to construct the ground to fourth floor of the HBB (the basement was constructed using insitu concrete). As mentioned previously, the precast planks are temporarily supported by props which are erected prior to installation of the planks. The precast planks act as permanent formwork and contain the bottom reinforcement for the floor structure. As the concrete frame progressed upwards, the floors constructed below the floor under construction were used to support the self-weight of the next floor. A temporary works engineer was employed by the contractor to determine a back propping sequence for the floor structure at each level as construction progressed. The back propping sequence must ensure that the load from the wet concrete is transferred to a sufficient number of floors below so that no slabs are overloaded. When the compressive strength of the insitu topping had reached a specified strength, the supporting props were removed and repropped so that each slab was supporting its own self-weight (also known as striking and repropping).

During the construction phase of the HBB, the changes in strain when props were dropped, back-propped and removed can be analysed, as well as when floors were poured above second floor. The deformation of the slab soffit during the construction phase was also recorded using a digital level (accuracy ± 0.6mm). The change in strain along a gridline (GL H) in the second floor for a series of VW strain gauges in the top of the insitu topping at two loading events is shown in Figure 10. The solid line is the change in strain when the third floor was poured (13th August 2015) and the second floor partially supported the newly cast floor above and the dashed line is the change in strain when the props were dropped and re-propped 15 days later (28th August 2015). When the third floor is poured, its self-weight is transferred down to the floors below (ground, first and second floor) using the system of backpropping. The recorded strains shows that when the props were removed from the third floor, so that it supports its own self-weight, that the imposed strains (and stresses) on the second floor from the third floor pour are reversed. The magnitudes of the change in strain when the third floor was poured when compared with the measured changes in strain when the second floor was poured suggest that the second floor supported approximately 20% of the self-weight of the third floor until the props were removed. The self-weight of the third floor is not equally distributed between the supporting floors below because of each slab has a different stiffness due to the sequence of construction and also the elasticity of the supporting props.

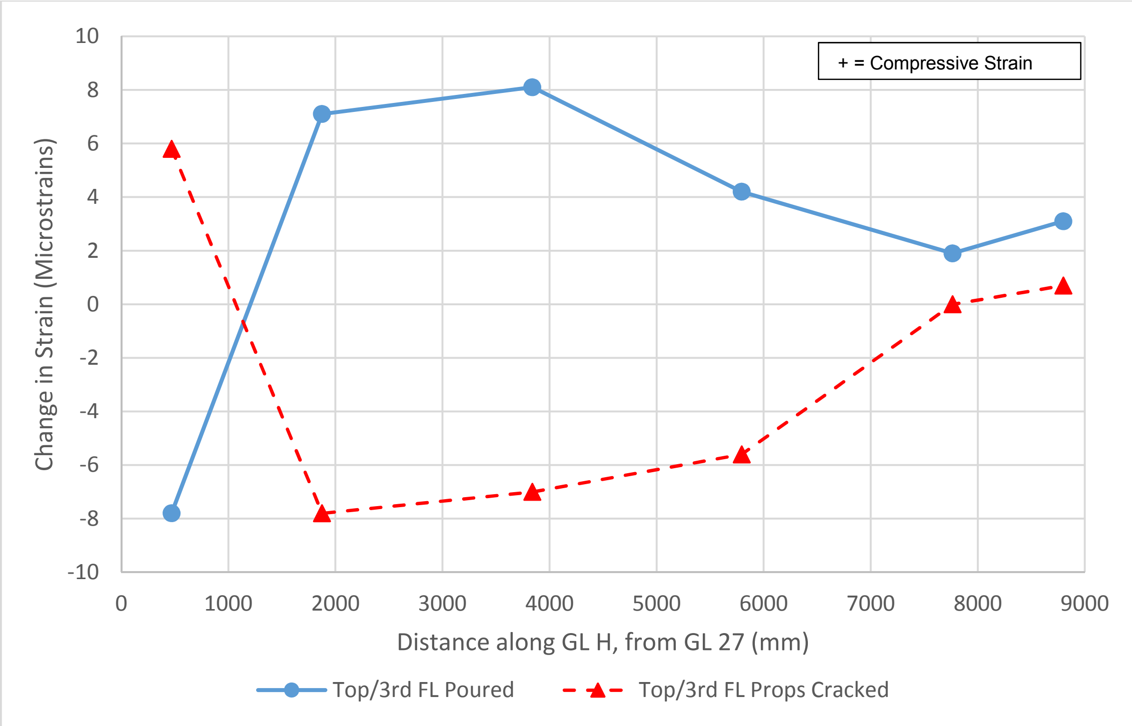


Figure 10. Measured change in strain in the second floor when (i) when the third floor was poured and (ii) striking of props

4.5. Strain profile

The measured changes in strain though the 400mm thick floor structure at one location along gridline H is shown in Figure 11 when the third floor is poured (13th August 2015) and when the props supporting the third floor are dropped (28th August 2015). At this location, there are three VW gauges in the insitu topping and one VW gauge in the precast plank. The strain profile illustrates the composite nature of the hybrid concrete floor structure and also that the slab appears to exhibit linear elastic behaviour at this stage of construction and could be considered 'uncracked'. Similar to the behaviour observed in Figure 10, the measured change in strains through the concrete floor in second floor when the 3rd floor is poured above are reversed when the props supporting the 3rd floor are removed.

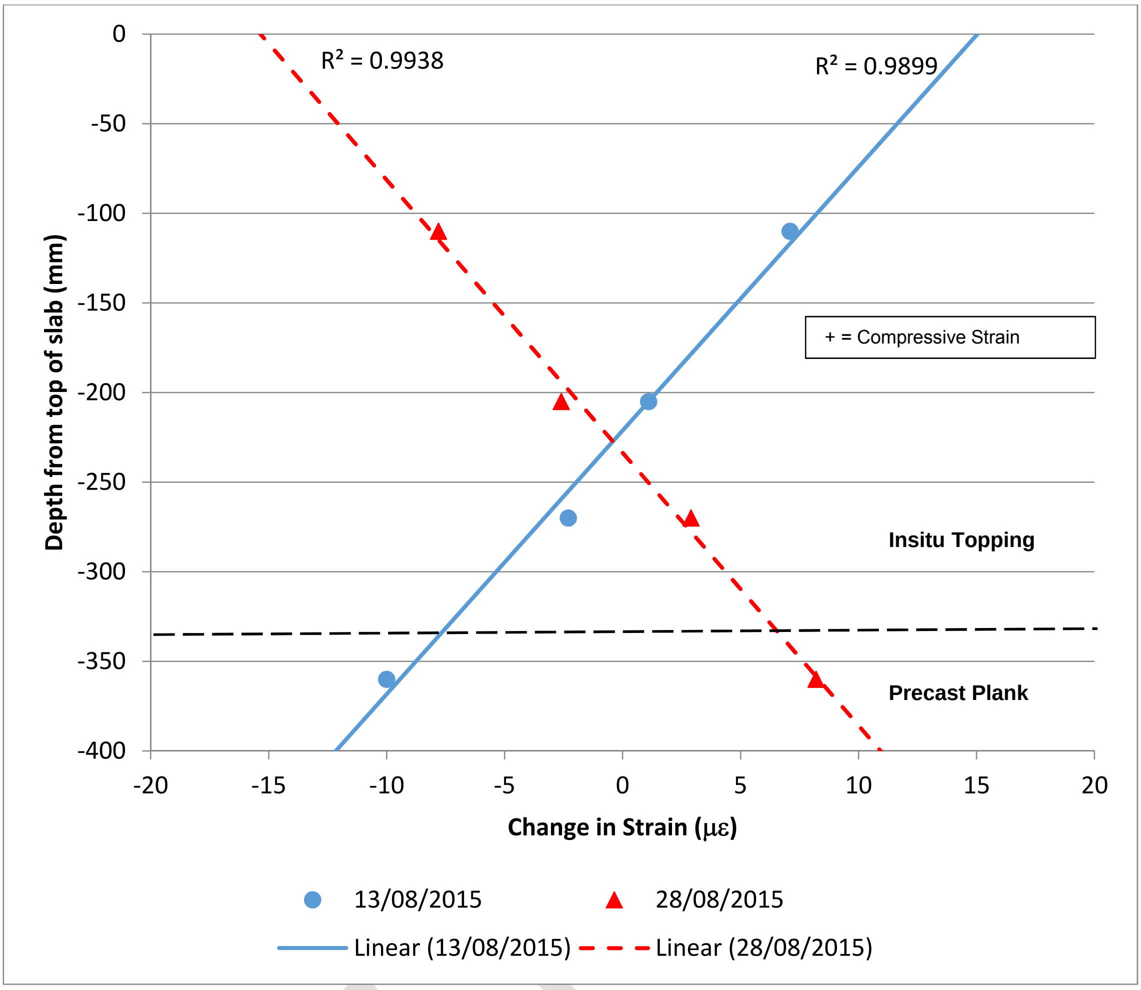


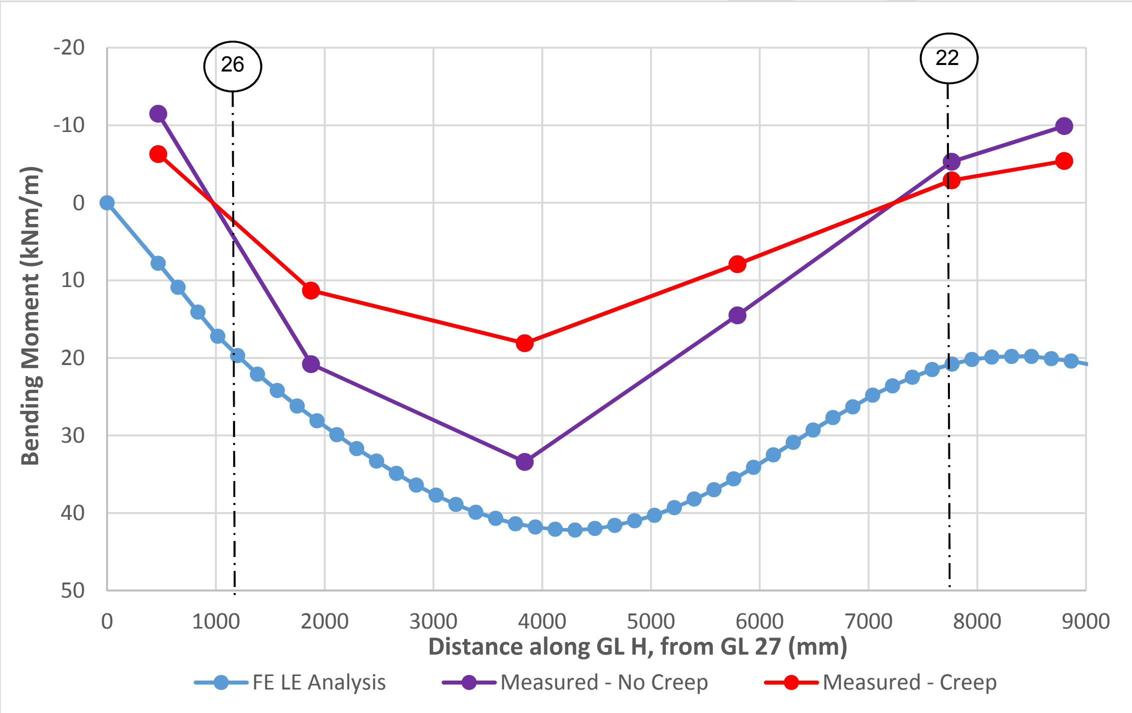
Figure 11. Measured change in strain in the second floor when (i) when the third floor was poured and (ii) striking of props

The measured change in strains from the embedded VW strain gauges after striking props to the second floor and repropping (to let slab support its own self weight) and when the props were removed permanently are assumed to be broadly equivalent to when the second floor supports its own self-weight. The applied load on the slab when it supports its self-weight is approximately 67% of the total design service load on the slab. The measured changes in strain along sections of the slab were converted to bending moments assuming the slab is uncracked (as suggested by measured changes in strain in Figure 11) and using the concrete properties derived from material testing conducted on the concrete used on site for the second floor. The concrete test results for the air-cured specimens were used when determining moments from the measured strains as they were cured in a similar environment to the concrete on site and are more representative of the insitu concrete in the floor slab. Although the specification for the insitu topping was C30/37 (f_{ck} = 30.0 N/mm^2), the average 28 day compressive strength (f_{ck}) from material testing was 36.5 N/mm^2 and 41.5 N/mm^2 for air cured and water cured specimens respectively (cylinders and cubes were tested). This illustrates one of the challenges of predicting behaviour of concrete structures at design stage, as typically assumed values for concrete properties will have to be assumed which may not be representative of the insitu concrete on site.

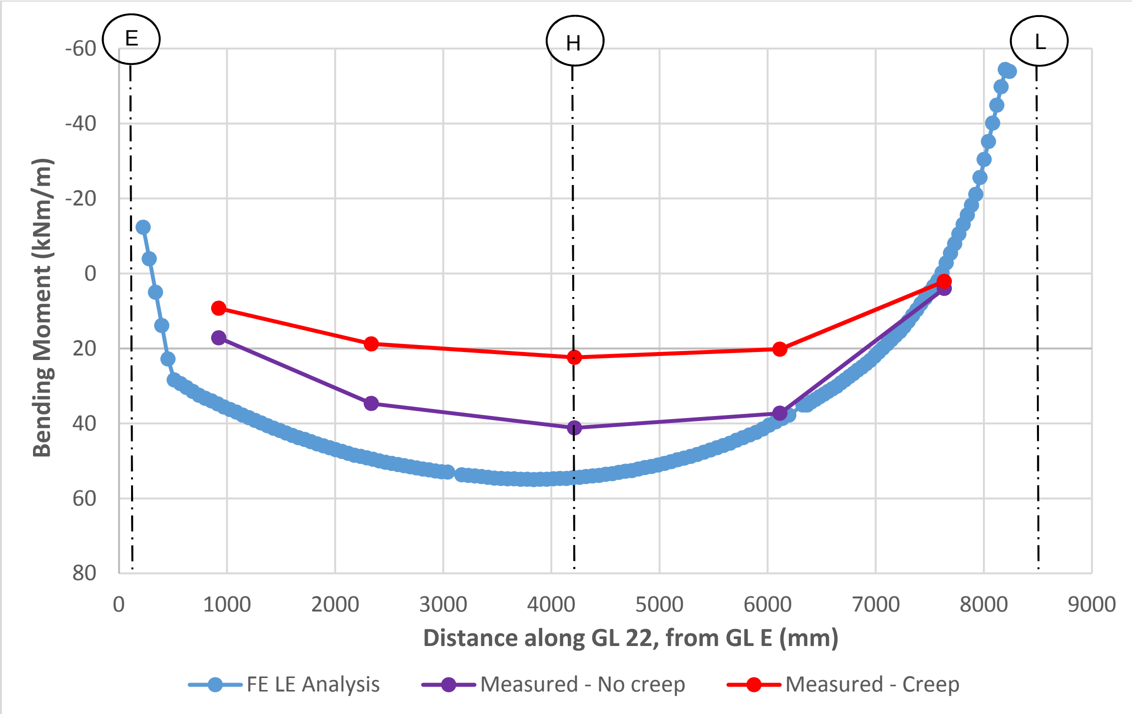
During the design process, a linear elastic finite element (FE) model was developed by the designers (precast manufacturer was responsible for the design of the hybrid lattice girder floor) in order to model the floor structure. In the analysis, the floor is modelled as 400mm thick insitu flat slab and the joints between the precast planks are modelled as a 65mm high and 130mm wide recess in the slab soffit. For the purposes of design, the material properties for the slab are assumed to be uniform throughout the depth and are taken as that for the insitu topping of the slab (C30/37, f_{ck} = 30 N/mm^2). The moments determined were compared with the moments from the model on second floor when it supports its own self-weight along gridlines (GL) H and 22 (Figure 5). The location of embedded sensors in the floor are noted in Figure 5 and typically consist of three or four VW gauges through the depth of the 400mm thick hybrid flat slab at each location as detailed in Figure 4 previously.

The bending moments determined using the measured strain data and the concrete material testing are compared with predicted bending moments from the linear elastic finite element analysis in Figure 12 for both gridlines H and 22 (orthogonal to each other). The measured strains were converted to bending moments for both the case of no creep and creep (creep coefficient determined in accordance with Eurocode 2). There is relatively good correlation between the predicted bending moments and moments derived using the measured strain from the insitu instrumentation and the comparison can be used to highlight differences between assumed and actual behaviour of the floor structure. The FE model is conservative (upper bound) as it does not take account that the 65mm thick precast plank was manufactured 25 days prior to the insitu topping and using a higher grade of concrete (C40/50) than the insitu topping (C30/37). Therefore, the actual floor slab is stiffer than modelled and we would expect that the moments derived from the measured strains would be less than the moments from the FE analysis. In addition, the model is based on a linear elastic behaviour and does not take account of creep in the concrete. Along GL H (Figure 12(a)), the insitu instrumentation records a hogging moment adjacent to the slab edge (GL 27) where the slab is supported by a reinforced concrete wall (twin-wall construction). In the FE model, the support condition was modelled as pinned and the bending moment is zero at the slab edge, but the embedded instrumentation indicates that the wall provides some torsional restraint. For this reason, many structural design codes require additional torsional reinforcement at slab edges to resist bending which may occur in real structures, but are not determined during analysis or where simply supported end conditions are assumed. If the ‘measured’ bending moment along GL H is shifted downwards by the magnitude of the hogging moment at GL 27, there is good agreement between the analysis and insitu instrumentation.

During the striking, repropping and permanent removal of props to the second floor, the walkway slab along GL 22 (250mm thick, hybrid concrete lattice girder slab) adjacent to the slab at GL L was continuously propped and, therefore, the predicted hogging moment over the column at GL 22-L from the FE analysis would not be as large in the actual floor structure. Similar to GL H, there is relatively good correlation for the sagging moment at midspan along GL 22 (Figure 12(b)), and as expected the 'measured' hogging moment at GL L is less than the predicted moment. The embedded sensors in the walkway slab did record moment transfer after striking the props for repropping and permanent removal of props in the adjacent 400mm thick slab and this would reduce the hogging moment over the column at GL 22-L. The difference between the predicted moment over the column at GL L and the 'measured' moment may also be due to the fact that FE software can over-estimate peak moments over columns depending on the mesh size and how the column is modelled.



(a) Along GL H



(b) Along GL 22

Figure 12. Predicted and measured bending moments in second floor of HBB; (a) Along GL H, (b) Along GL 22

4.6. Time dependent effects

The prediction of strains for concrete elements systems is extremely difficult to estimate accurately (Gilbert, 2001). The behaviour of a slab at service loads varies with time and depends on the extent of cracking, stiffness of the slab, creep, shrinkage and degree of restraint. A range of +15% to -30% between calculated and actual deflections is suggested in The Concrete Society publication on deflections in concrete slabs (The Concrete Society, 2005). One of the key goals of this project is to monitor the time dependent effects (TDE) on the slab in HBB through the use of the insitu instrumentation. In this project, data will continue to be recorded from the sensors after the building is operational so that the long-term behaviour of the slab can be analysed and compared with various predictive models. There are numerous predictive models in the literature and design codes (BSI, 2004; ACI, 1992; Gilbert and Ranzi, 2010) for predicting TDE but there is still a large degree of uncertainty for predicting accurately the long-term behaviour of concrete structures at serviceability limit states. One of challenges when predicting TDE is that the properties of concrete change with time and at design stage it is very difficult to accurately know the insitu properties of the concrete as they evolve over time in response to environment and loading.

The slab in the HBB is subject to strains from thermal, shrinkage, creep and flexural components. The predicted strains in the second floor slab due to the various components determined using Eurocode 2 (BSI, 2004) are compared with the measured strains from an embedded VW gauge at midspan along GL 22 in Figure 13 for the first six months (June 2015-Dec. 2015 inclusive) after the floor was poured. The predicted strains are determined using assumed values for the concrete properties and relative humidity. The applied loading due to self-weight and creep are assumed to commence after striking the props and repropping, which was 8 days after the insitu topping was poured. The measured strains from the VW strain gauges are corrected for thermal effects so that only strain due to shrinkage, flexure and creep is determined from the gauge readings:

$\mu\varepsilon_{load} = \mu\varepsilon_{actual} - \mu\varepsilon_{thermal} = \Delta\varepsilon + \Delta T.\alpha_{vw} - \Delta T.\alpha_{c}$

3.

where $\Delta\varepsilon$ is the change in measured strain; ΔT is the change in temperature; α_{vw} is the coefficient of thermal expansion of the VW gauge and α_c is the coefficient of thermal expansion of the concrete.

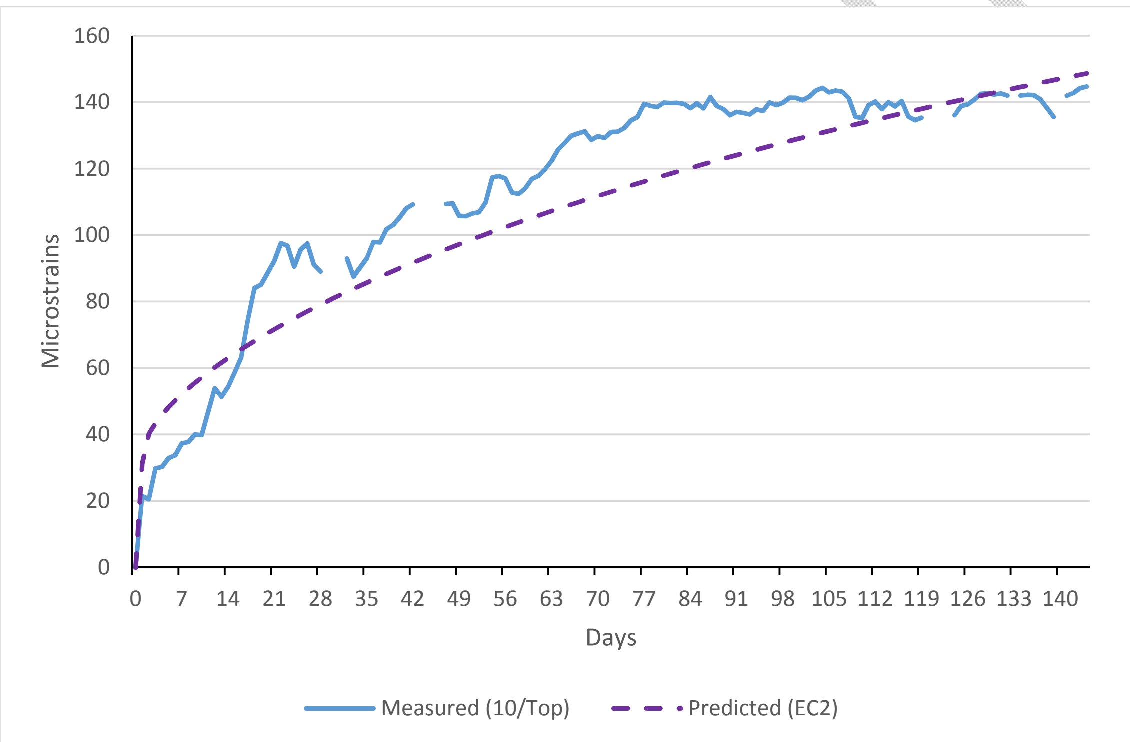


Figure 13. Comparison of predicted and measured strains in second floor of HBB

The dominant strain component is shrinkage and this accounts for approximately 60% of the strain induced on the second floor slab. For the strain gauge shown in Figure 13, there is relatively good correlation between the measured and predicted strain values considering the relatively large coefficient of variation for both creep (20%) and drying shrinkage (30%) when using the approach of Eurocode 2 (BSI, 2004) for determining strains. The next stage in this process is to incorporate actual concrete properties determined from testing and environmental data from the NUIG weather station (RH and temperature) and internal sensors to compare the effect of using assumed design values and actual data when calculating predicted strains. As there are over 100 VW gauges embedded in the second floor slab, it will allow a meaningful study of the correlation between measured and predicted strain using a variety of models. The data from the sensors will continue to be collected from the sensors so that TDE in the floor slab (such as creep) can be studied and compared with predictive models.

5. Practical applications

The use of SHM, as implemented in this research project, has considerable practical relevance for Structural Engineering practitioners and many potential applications. Design guidelines are typically approximations which simplify the behaviour of the structure so that it can be analysed relatively quickly and simply. However, these approximations and simplifications have the potential to introduce errors or inaccuracies to the design process. The insitu instrumentation provides performance data on actual behaviour which can be used to verify design methods. Real time monitoring offers potential benefits in relation to optimisation of structural components by understanding the actual behaviour of components in use and the possibility to develop and calibrate numerical models that predict structural performance.

This paper illustrates the important role which real time monitoring (SHM) and the use of sensors can play for more accurate prediction and understanding of the actual structural behaviour of concrete components. Some of the practical applications demonstrated in this paper include:

- The use of sensors to accurately predict the temperature rise and temperature differentials in concrete components which can lead to more accurate predicting of early-age behaviour.
- The significance of accurate knowledge of material properties and environmental conditions to predict precisely early-age thermal cracking. The paper shows that basing design decisions on theoretical properties or assumed conditions can potentially lead to inappropriate design decisions.
- The determination of the restrained insitu coefficient of expansion in concrete components leading to more realistic restraint factors. Incorrect restraint factors can result in uneconomical over-design or under-design leading to unacceptable cracking.
- The striking of formwork and backpropping is a critical part of the efficient and economic construction of flat slab structures. Embedded sensors can be used to measure the actual load transfer mechanism and assist with improved temporary works design and more efficient construction techniques.
- SHM can be used to improve understanding of actual structural behaviour and information from SHM programmes can be used to improve prediction models and codes of practice. In addition, with the increasing use of new construction materials and innovative construction practices, sensors can used to provide confidence for designers in relation to actual performance of structures.

- Predicting time dependent effects (TDE) in concrete is one of the most difficult tasks in structural engineering. The use of sensors as described in this research project can be used to study the relationship between measured strain and the various prediction models.

6. Conclusions

This paper presents the motivation and implementation of a real time SHM strategy for a hybrid concrete lattice girder flat slab floor system. The embedded sensors in combination with the material testing and weather monitoring allow the performance of the floor structure to be monitored and the effect of various factors such as temperature, relative humidity and construction method to be studied. This project is such that data is recorded during the manufacture, construction and operational phase of the building so that early-age performance, construction behaviour and long-term effects can be analysed. It is envisaged that the rich information about the performance of the floor can be used to study the various design parameters of the hybrid concrete lattice girder flat slab floor system and identify potential optimisation of the floor system.

Discrepancies between performance of construction materials in the laboratory and their observed performance in the field has been reported (Gulis, Lai and Tharambala, 1998), particularly with respect to serviceability limit states of concrete (durability, deflection etc.). Therefore, insitu instrumentation can be used to assess the actual performance structural elements and assist designers during the design process of concrete structures to achieve long-term in-service performance criteria.

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