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Evaluating the structural capacity of concrete elements through in-situ instrumentation

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Abstract. The difficulty in predicting the long term load capacity of concrete elements is well documented. Time dependent effects such as creep and shrinkage coupled with varying loading events, particularly during construction, can all have an adverse effect on the long term performance of a concrete structure. This paper proposes a method that utilises in-situ instrumentation to predict the load carrying capacity of concrete members. During the construction of the Engineering building at the National University of Ireland, Galway over 260 sensors were embedded in a number of key concrete elements. The sensors are being continually monitored with the use of automatic datalogging equipment and the data is being used to monitor changes in geometric and material properties along with the subsequent time dependent deterioration of the elements. The paper will illustrate how the in-situ data from the demonstrator building can be used to estimate the real time behaviour of the concrete elements and how these elements might respond to future changes in use and potential retrofitting. A cost analysis will show how such a monitoring system can be used to reduce the uncertainty levels involved when retrofitting concrete buildings.

Introduction

The behaviour of concrete structures during the ultimate state is well understood. This is primarily due to research which has been carried out over the past number of decades. However, during the life of a concrete structure there are a variety of mechanisms which can affect the behaviour of the structure, both in serviceability and ultimate limit states. One of the many problems lies in the inherent make-up of concrete itself. Unlike other structural materials concrete properties are continually evolving over time. Its compressive strength, tensile strength and modulus of elasticity all change in response to environmental and loading conditions. In addition complex time dependent phenomena such as creep and shrinkage are often the main influence on internal actions and have been established as one of the main causes of non-structural cracking. These effects have proven notoriously hard to predict with numerous studies carried out to try and establish them [1-3]. Cracking of course has an adverse affect on the concrete structure resulting in a change of the corresponding stiffness and the internal stresses experienced within the element. Deterioration due to time-dependent effects within the structural elements may result in reduced capacities. Furthermore the effect of construction loading has been shown to affect the capacity of concrete structures [4-5]. As such an accurate method to establish the in-situ behaviour of a concrete element would be a valuable tool when assessing structures. In an effort to monitor the in-situ behaviour of concrete structures a series of sensors were installed within a number of concrete elements during the construction of the new Engineering building (EB at the National University of Ireland Galway (NUIG)). The idea forms part of an overall plan to develop

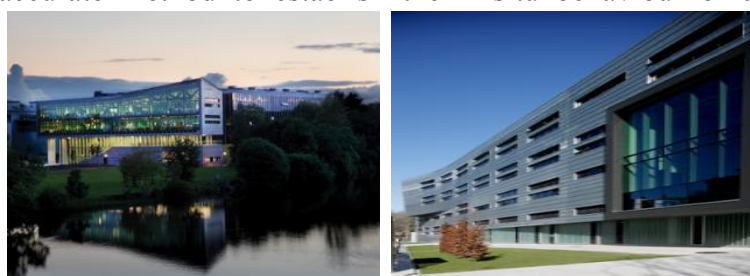


Fig. 1: The new Engineering building at the National University of Ireland Galway.

the building (Fig. 1) as a 'living laboratory' for students [6]. The instrumentation of the concrete elements makes it possible to estimate when the reinforcement and concrete will reach their yield capacity and represents an important tool in helping to increase the service life of the concrete elements.

Instrumentation of concrete elements at NUIG

At 14,250m² the EB at NUIG is the largest engineering school in Ireland. The building united all engineering disciplines within the university into a state of the art academic facility. Previous papers [6-8] have illustrated the overall instrumentation of the building and its development as a research tool. As such only a brief summary will be given in this paper.

To collect data in relation to the behavior of concrete elements a total of three concrete members were instrumented during the construction of the EB; namely a prestressed double tee beam, a prestressed transfer beam and a void form flat slab. Vibrating wire (VW) strain gauges (Gage Technique model TES/5.5/T) were embedded in the concrete elements to monitor both temperature and longitudinal strains. In addition to the VW gauges a number of electrical resistance gauges were also installed on the reinforcement bars in the void form flat slab.

Prestressed Beams

The prestressed transfer beam contains a total of 52 VW gauges distributed over 7 sections, five of which are grouped around a concentrated load near midspan, while the other two sections are located near the supports. Most sections contain six gauges; two VW gauges at the top, middle and bottom of the section. Some sections contain additional gauges with the intention of extracting more detailed data. The prestressed beam is 16.75m in length, 970mm wide by 1200mm deep. Critical members within the EB have been designed to cater for an additional floor in the future. The concentrated load acting on the prestressed beam is essentially a support column for one of the upper floors and as such the element will play a critical role in assessing the potential retrofitting of the building in the future. In addition to the prestressed box beam a prestressed double-tee unit was instrumented with a total of 39 VW gauges; 13 at gauges at 3 separate sections. The final concrete element instrumented within the building was a void form flat slab system and the analysis of the results from this element form the basis of this paper.

Void-form flat slab

The void form flat slab has a total of 64 vibrating wire gauges installed over 15 designated sections. The slab has a long span of 12.65m, a short span of 7.5m and an overall thickness of 450mm. The layout of the sensors installed in the slab can be seen in Fig. 2. To monitor strains in the east-west direction three vibrating wire gauges were positioned in each location - one at the top, middle and bottom of the slab. Two vibrating wire gauges were used in each of the same locations to monitor strain and temperature in the north-south direction, with gauges located at the top and bottom of the slab respectively. The bay in question was poured in June 2010 and strain and temperature profiles have been

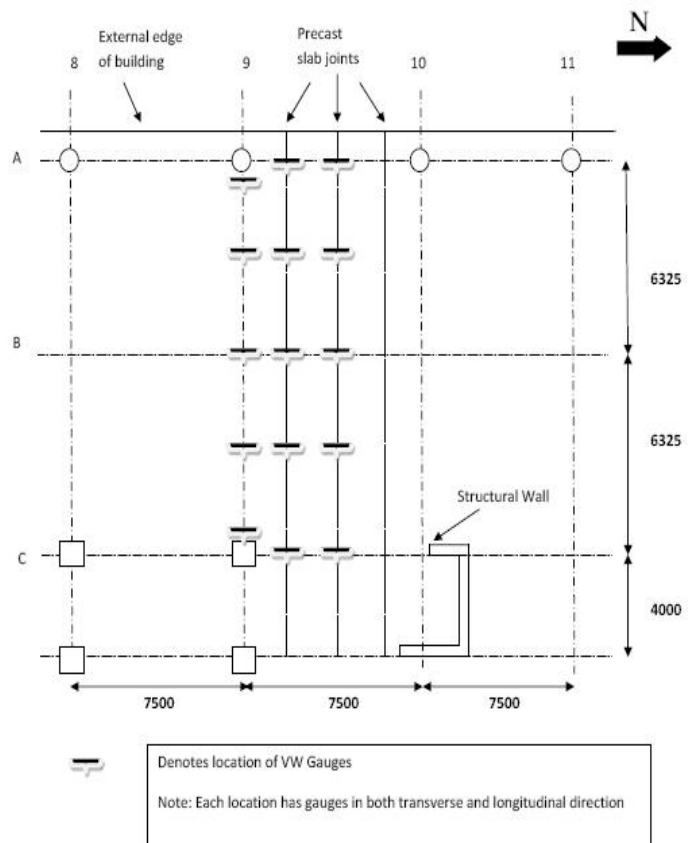
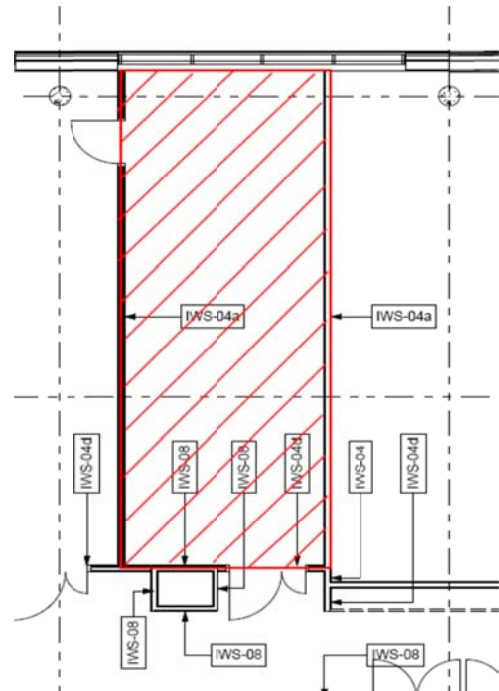


Fig. 2: Plan view showing layout of VW gauges in the void form flat slab.

continuously monitored since. In addition to the VW gauges 100 electrical resistance gauges were installed on steel reinforcement bars at 5 different locations along the east-west centerline of the slab bay. As such it is possible to determine relationship between strains in the reinforcement bars and corresponding VW gauges located at each of the 5 sections.

Potential retrofitting and strengthening of void form flat slab

As a means of illustrating the techniques in estimating the load capacity of the instrumented flat slab element the functionality of one of the rooms supported by the slab will be changed. This can occur quite often in buildings when a situation may arise where the structural system is subjected to higher loads and displacements than those considered during the original design stage. The room to be changed is approximately 47.5m^2 in area and can be seen (shaded) in Fig. 3. The instrumented slab was initially designed to cater for dead loads (in addition of self-weight) of 1kN/m^2 and a live load of 6kN/m^2 . The room will subsequently be designed to cater for a similar dead load arrangement but with a new live load of 11kN/m^2 . When such cases arise in reality there may be a need to strengthen the flexural capacity of the concrete member. For flat slabs the increase in load will typically result in flexural deficiencies in the negative and positive moment regions of the member. In recent times the most popular method of providing additional flexural strength to concrete elements is through the use of fibre reinforced polymers (FRP's) bonded externally to the concrete surface in the direction of the existing reinforcement, parallel to the direction of the tensile stresses [9]. The most common FRP materials used include carbon (CFRP), glass (GFRP) and aramid (AFRP). The paper will present estimates of the load carrying capacity of the slab based on guidelines given by the *American Concrete Institute; Design Report 440.2R-02* [10] and *FIB Bulletin No.14* [11]. In addition an alternative load capacity method will be illustrated utilising data from the



instrumentation of the void form flat slab. Design calculations were carried out for all three FRP materials mentioned above and a cost analysis produced to see which material and design method would result in the lowest construction cost.

Estimating structural capacity of concrete elements

The strengthening and design of members in flexure using externally bonded FRP's is based on limit state principles and relies upon the composite action between the concrete section and the externally bonded FRP's. It is typically carried out using the strain compatibility reinforced concrete flexure theory (Fig. 4). The theory is also the most common method used in estimating the structural capacity of concrete sections. It relates to the fact that for reinforced concrete sections, stresses in the steel are a function of those acting within the concrete. Essentially a stress resultant is calculated for the force in the compression steel, the force in the concrete compression block and finally the force in the tension steel (the force in the concrete tension block is ignored due to the low tensile capacity of concrete). Moments of all these forces are calculated about the neutral axis of the section. For the section to be in equilibrium this moment must be equal to the applied bending moment.

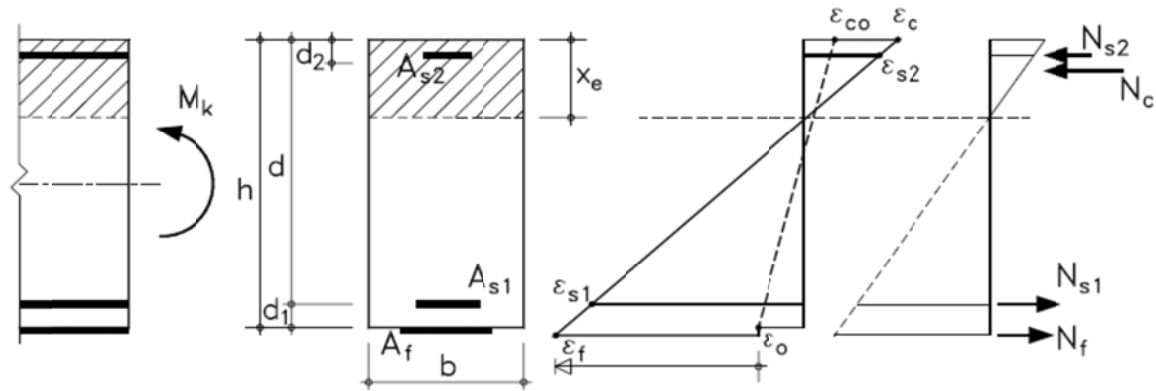


Fig. 4: Illustration of strain compatibility theory used in the estimating the structural capacity of concrete sections and in the design of externally bonded FRP's [10].

The strain compatibility theory is the approach adopted by both ACI 440 [10] and the FIB Bulletin 14 [11]. Both methods are similar in many aspects of the design procedure adopting a typical linear strain distribution in the section. They both require verification of conditions relating to bond, effective stress levels in the steel and FRP in both the serviceability and ultimate limit states. However they do differ in some aspects. ACI 440 [10] assumes a maximum compressive strain in the concrete of 0.0030 while FIB Bulletin 14 [11] assumes a value of 0.0035. Furthermore they differ in the manner in which they apply strength reduction factors to the FRP materials. ACI 440 [10] uses a strength reduction factor of 0.85 in addition to an environmental strength reduction factor which is dependent upon exposure conditions and on the type of FRP chosen (i.e. carbon, aramid or glass). FIB Bulletin 14 [11] uses FRP material safety factors for each individual material depending on the application type of the material (i.e. CFRP = 1.20, AFRP = 1.25, GFRP = 1.3). Details of the FRP material properties used in the retrofit design can be seen in Table 1.

Table 1: FRP laminate properties used for flexural strengthening of the void form flat slab.

	E - Modulus [N/mm ²]	Tensile Strength [N/mm ²]	Strain at Rupture [mm/mm]	Ply thickness [mm]
CFRP (SikaWrap Hex 103-C)	65,087	717	0.0113	1.016
GFRP (SikaWrap Hex 100-G)	20,044	484	0.0231	1.016
AFRP (SikaWrap 300-A)	23,000	440	0.028	1

The strain compatibility theory needs to accurately establish the section properties (neutral axis depth, second moment of area etc.) for the concrete section for both the cracked and uncracked states. In addition concrete material properties (i.e. compressive strength, tensile strength, modulus of elasticity, creep coefficient, creep curves) need to be established. Perhaps the most critical aspect when designing for externally bonded FRP is establishing the existing strains on the soffit of the slab. The amount of FRP required is directly related to these strains and is determined analytically by equalising the tensile force that would have been utilised by the steel reinforcement, which has now yielded, with the tensile force that will now be utilised by the additional FRP reinforcement. An important consideration to make when calculating externally bonded FRP reinforcement for concrete structures is the lack of plasticity in the FRP components. As such the redistribution of moments in the strengthened member, a common practice in reinforced concrete design, is not permitted.

From the instrumentation of the concrete elements in the EB it is possible to accurately determine the initial strains acting on the soffit of the concrete section, not just at critical locations in the elements (i.e. mid-span and at supports) but at each individual sensor location thus allowing for optimisation of any possible FRP layout. The moments acting in the void form flat slab were calculated, for both ACI 440 [10] and EC2 [12] load factors, using finite element analysis. The moments utilised in determining the strength capacity relate to unfactored moments from dead loads at the time the FRP is applied. Section properties were calculated from the formulas given within the respective design guidelines [10-11] and these were compared with separate calculations carried out using the computer software package *Oasys AdSec*. Section properties along the longitudinal column strip can be seen in Table 2. These were derived in accordance with both ACI 209 [13] and BS EN 1992-1-1 [12] guidelines. To complement the instrumentation process a detailed material testing programme was carried out to establish properties such as the compressive strength, modulus of elasticity, tensile flexural and splitting strengths of the concrete mix. Perhaps the most critical properties, in terms of designing the externally bonded FRP, are the compressive strength and modulus of elasticity. The concrete grade used in the construction of the slab element was C28/35 in accordance with EC2 [12]. Although prediction models for the compressive strength were in line with prediction models outlined within the design codes [12-13] the modulus of elasticity was found to be almost 10% higher than models outlined within the EC2 [12] and over 20 % higher than those outlined by ACI [13]. This higher modulus helps to reduce strains in the steel reinforcement and also the externally bonded FRP but can does lead to higher stresses.

Table 2: Outline of section properties along longitudinal column strip.

Location	I_g (mm^4)	I_{cr} (mm^4)	ρ_s (%)
GL 9-A	8.98×10^9	4.123×10^9	1.25
GL 9-AB	7.82×10^9	3.47×10^9	1.36
GL 9-B	7.82×10^9	3.47×10^9	1.36
GL 9-BC	8.43×10^9	3.07×10^9	1.93
GL 9-C	9.32×10^9	4.28×10^9	1.75

I_g = Gross second moment of area of concrete section; I_{cr} = Cracked second moment of area of concrete section
 ρ_s = Percentage of reinforcement

Creep Effects

Creep is also an important consideration to take into account when retrofitting concrete structures. It is well documented that concrete undergoes creep when subjected to a sustained stress. The same is true for some FRP materials. It is possible for sustained loads to induce creep in the fibers that can ultimately result in rupture of the fibers. Research has shown however that CFRP does not creep and that carbon fibers are capable of resisting 80% of their ultimate strength over time. Creep in GFRP has been found to be negligible but the creep effect can in AFRP sections can be critical [14]. As such the creep of both CFRP and GFRP is primarily determined by the compressive creep of the concrete and allowance for creep rupture of AFRP is accounted for in the design guidelines. However creep in the concrete can be critical. Recent research [15] has shown that predictive models within design codes, specifically the FIB models [16] underestimate the effect of creep. As the concrete creeps it results in a lowering of the neutral axis thus changing the effective properties of the section and as such the strain on the soffit of the slab [17]. As previously mentioned, the various types of fibers have varying resistance to creep effects. The authors previously explored the effects of time-dependent effects in concrete sections using in-situ measurements [18]. Strains resulting from time-dependent effects were modeled using the theory of age adjusted elastic modulus [19] and creep transformed sections as outlined by Ghali [20].

Agreement was found with prediction models but that the creep strains experienced within the instrumented slab were higher than those outlined within both EC2 [12] and ACI 209 [13]. Using these models it is possible to calculate the predicted strain on the soffit of the slab due to creep effects from the increased loading when calculating the externally bonded FRP reinforcement.

Cost analysis

A cost analysis was carried out to illustrate the benefit of utilising instrumentation data when estimating structural capacity and the subsequent retrofitting of structures. Costs were calculated based on the quantities of externally bonded FRP required for each of the design situations. Cost ratios relating to the type of FRP used are based on the work of Burgoyne [21].

Results

The structural capacity of the slab was calculated using the principles outlined in ACI 440 [10] and FIB Bulletin 14 [11]. Values were also found by substituting the calculated in-situ soffit strains from the instrumented slab for their corresponding values in the aforementioned design guides. Suitable externally bonded reinforcement arrangements were calculated for each of the FRP materials mentioned earlier; namely CFRP, GFRP and AFRP. Externally bonded FRP reinforcement was found to be required around the area of each of the column heads in both the longitudinal and transverse directions and again at the mid-span of the slab. It was found that the layout and shape of the external FRP reinforcement were quite similar for all design solutions. This is primarily due to the simplified detailing requirements outlined within the design guidelines [10-11]. However there was a considerable reduction in the amount of FRP (i.e. no. of layers etc.) required when utilising the data from the instrumented slab. In-situ strains were almost 20% less than those calculated using the design guidelines. However it was found that strains from the in-situ instrumentation were larger in the portion of the slab supported by the structural wall (see Fig. 2). Overall the ACI 440 [10] design procedure required the most externally bonded FRP reinforcement, followed by the FIB Bulletin 14 [11] (4 % less) and the least amount of reinforcement established using the in-situ data (15 % less). These quantities are obviously directly related to the total cost of strengthening the structure. Results (Fig. 5) show that the most economic solution is to use CFRP. CFRP is the most expensive per unit; however its high strength reduces the quantity needed. GFRP proves to be the second most economical solution. It is the cheapest of the three materials. However its low strength, in comparison to CFRP larger quantities are needed. AFRP is the least desirable in economic terms. Its cost is comparable to that of CFRP however the large quantities needed result in an uneconomic solution.

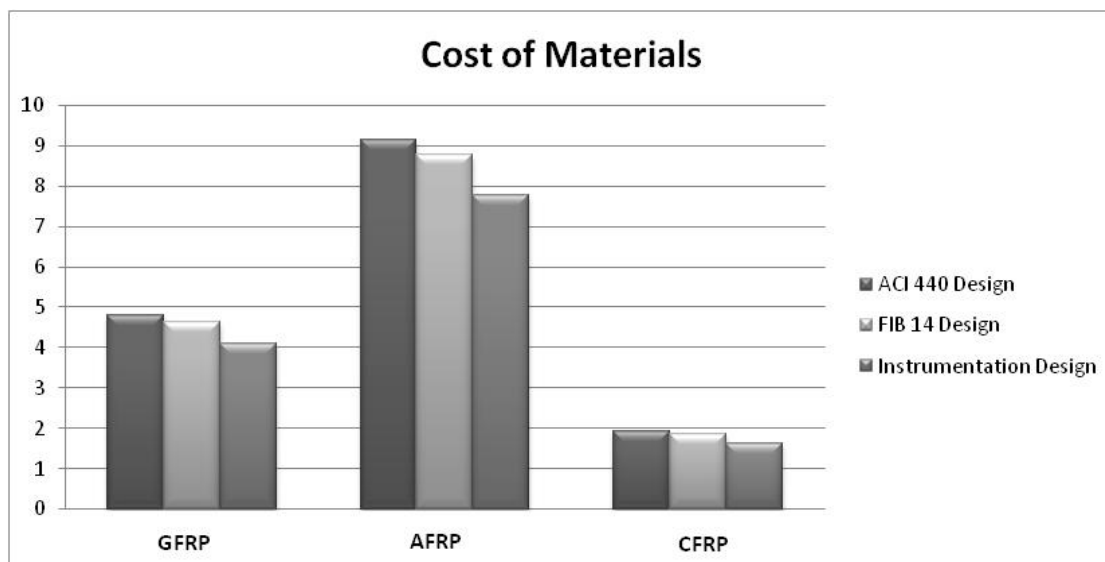


Fig. 5: Graph showing comparative unit cost of materials for retrofitting of void form flat slab.

Consideration was also made with regard to creep effects in the concrete due to an increase in load. Different time periods for when the load would be applied were analysed with regard to FRP reinforcement quantities and cost. It was found however that the effects of creep were minimal with only a slight increase in strains over time and not enough to excessively affect the amount of externally bonded FRP reinforcement required. This is primarily due to the fact that creep in concrete is strongly related to the time at which load is applied to the concrete element. The longer the period before load is applied the less the concrete will creep.

Conclusions

Based on the evaluation of the cost analysis it is evident that utilising the data from the instrumentation of the void form flat slab offers a more refined design approach than those outlined within the design guidelines [10-11]. This paper has presented a novel way of estimating the structural capacity of reinforced concrete structures. By using in-situ instrumentation it is possible to accurately estimate the capacity of concrete elements which can prove extremely useful when retrofitting and strengthening structures. By using the design principles outlined in relevant design codes and guidelines and substituting the in-situ strains from instrumentation savings can be made in the quantities needed to carry out a suitable retrofitting scheme. Greater accuracy was also achieved by carrying out a comprehensive study of the material properties of the concrete grade. This is an important step as it was found the lower strain readings from the instrumentation process were offset somewhat by the higher modulus values, increasing the stress in the reinforcement at serviceability and ultimate states. The instrumentation data will also play a key role in facilitating the monitoring of damage progression and any change in load paths within the structural elements following the strengthening procedure.

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