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THE EFFECT OF LOCALIZED LOW CONCRETE STRENGTH ON FLEXURAL STRENGTH OF RC BEAMS

Feng Xie[1] and Riyad Aboutaha[2]

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In reinforced concrete (RC) beams, localized low concrete strength may occur under certain conditions, e.g. poor construction practice that results in concrete honeycombing. The performance of beams with localized poor zones has received considerable attention in civil engineering research. This report presents the response of beams with various localized poor zones along the length of simply supported flexural members. A finite element model was developed and calibrated against several experimental beam test data, conducted by others. To simulate localized concrete degradation effect, the concrete strength at different locations was reduced. To investigate the location effect, the beam was divided into three major regions, one was sensitive to bending moment, one was sensitive to shear, and the third region was sensitive to bond slip. The variables investigated under this study also included four types of concrete strength and three different rebar sizes. A total of 30 FEM beams were investigated. Based on the 30 FEM results, an empirical model was proposed to take the honeycombing location, rebar size, span length, and localized concrete strength into consideration. This data based empirical model can be used to approximately predict the effect of the localized concrete deterioration. An accurate practical model regarding to specific project can also be developed following the generalized approach introduced in the paper.

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ABSTRACT

In reinforced concrete (RC) beams, localized low concrete strength may occur under certain conditions, e.g. poor construction practice that results in concrete honeycombing. The performance of beams with localized poor zones has received considerable attention in civil engineering research. This report presents the response of beams with various localized poor zones along the length of simply supported flexural members. A finite element model was developed and calibrated against several experimental beam test data, conducted by others. To simulate localized concrete degradation effect, the concrete strength at different locations was reduced. To investigate the location effect, the beam was divided into three major regions, one was sensitive to bending moment, one was sensitive to shear, and the third region was sensitive to bond slip. The variables investigated under this study also included four types of concrete strength and three different rebar sizes. A total of 30 FEM beams were investigated. Based on the 30 FEM results, an empirical model was proposed to take the honeycombing location, rebar size, span length, and localized concrete strength into consideration. This data based empirical model can be used to approximately predict the effect of the localized concrete deterioration. An accurate practical model regarding to specific construction project can also be developed following the generalized approach introduced in the paper.

Introduction

Reinforced concrete structures are widely used over the world due to its cost-effective benefit. And reinforced concrete structures can work well as designed because the behavior of reinforced concrete structures could be easily predicated if constructed properly and well maintenance. However, poor construction practice could cause problems, such as honeycombing in concrete, and create zones of low concrete strength. For instance, many of the bridges built in United States are reinforced concrete bridges. But one in nine of the nation’s bridges are rated as structurally deficient [1]. To avoid unexpected failure of these concrete structures, we need to investigate the effect of localized low concrete strength on flexural strength of RC beams and the safety performance of those localized deteriorated RC beams.

Localized Poor Concrete Problem - Concrete is a manufactured material that has two types of construction methods, cast-in-place and precast concrete. Precast concrete is built in the factory and transferred to the construction site for installation in final position. As it is built in factory, it has stable physical characteristics and few defects. On the other hand, the quality of cast-in-place concrete can be affected by the various factors related to site

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condition that affect the quality of the concrete. Some construction errors are trivial and can be ignored or easily fixed. But the others which affect the strength of concrete should be investigated and fixed. The typical defects that will cause localized low strength concrete problems are listed below: (1). Excess concrete mix water - Concrete is a composite material consists of cement, aggregate and water. The cement water ratio could significantly affect the strength of the concrete. Sometimes the worker will add excessive water when it’s difficult to pour concrete. Although excessive water will increase the flowability of concrete, it will also reduce the concrete strength, increase the porosity and creep of concrete, and reduce the abrasion resistance of concrete [2]. (2). Poor vibration – they can cause honeycomb and rocket pocket, whose concrete mortar failed to bond the aggregates and leave voids inside the concrete due to lack of vibration or poor construction practice, which results in localized low strength concrete zones. (3). Form failure – When constructing cast-in-place concrete, forms have to be set up firmly before pouring concrete. If the forms are not properly set up and sealed, then the mortar would leak through formwork joints, which creates inferior concrete zones. (4). Finishing defects - after the concrete has been poured, construction work has to flatten the finish of concrete. During the finishing procedure, they will add some water to the surface of the concrete. This will generate a porous permeable and low strength concrete area. On the other hand, a poorly finished concrete surface is susceptible to premature spalling.

Reference Experiment

A recent experiment done by Lim Hwee Sin (2011) was chosen as reference experiment. In his research, the performance of lightweight concrete was carefully investigated. Although several simply supported beam were investigated experimentally, in the reference paper, beam No.1, the most simple and symmetric case, was chosen as the reference specimen to compare with the result of finite element model. The dimensions and detailing of the reference beam are illustrated in Fig.1. The section of the beam is 300mm high and 150mm wide. The span length of the beam is 2800mm from support to support. The concrete strength is 42.7 MPa throughout the beam. Eventually, the beam failed in flexural.

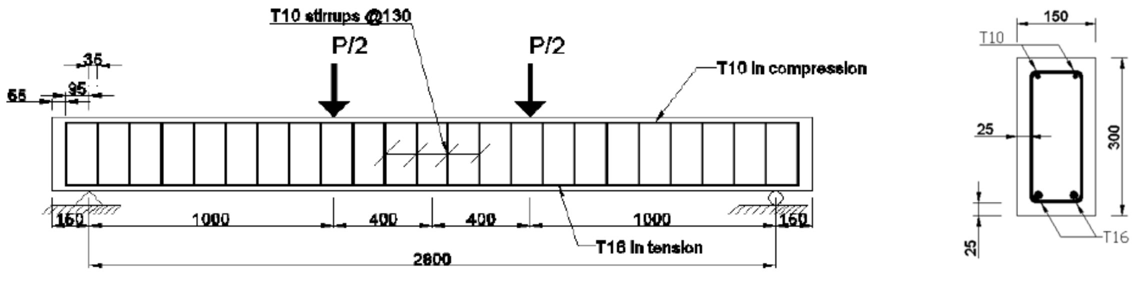

Figure 1. Details of flexural failure experiment [3].

Bogdan A. Podgorniak-Stanik (1998) has done an extensive experiment program to investigate the shear behavior of reinforced and prestressed concrete members. Case BN50 of his was used in this paper to testify the veracity of the finite element method when it comes to shear failure with splitting cracks. The dimension and detailing of the beam are illustrated in Fig.2. The beam has no stirrups due to the objective of investigating the shear failure in the experiment. Only tension reinforcement are provided here, they are two 20M rebar at the bottom corner and one 25M at the bottom middle. The depth of the beam is 500m, including 50mm concrete cover in both vertical and horizontal direction. The width of the beam is

300mm and the span of the beam is 2700mm. Supports are placed 150mm away from the end of the beam at each side. The concentrated load is applied in the top middle of the beam. Eventually, the beam failed in shear with some splitting cracks along the longitudinal reinforcement, which indicates a weak bond problem.

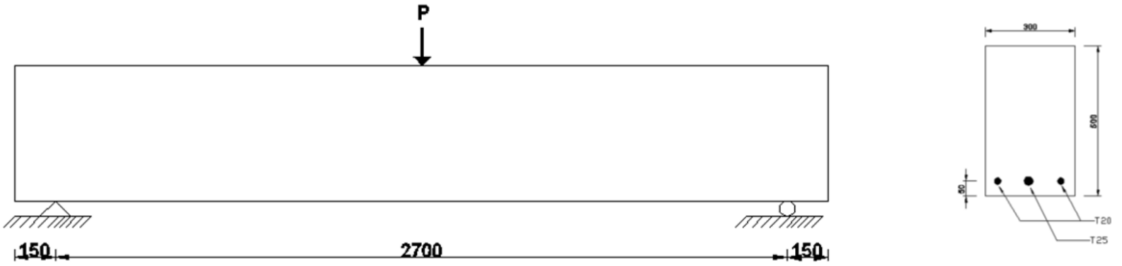

Figure 2. Details of shear failure experiment [4].

Finite Element Modeling

Materials

Concrete - SOLID65 was used to model the concrete. Concrete behavior followed the multilinear isotropic hardening using Von Mises theory. For the computational efficiency, the stress-strain law of concrete adopted a modified hognestad concrete model. The model is illustrated in Fig.3. And the failure criteria of concrete elements were modified Willam and Warnke criterion [5].

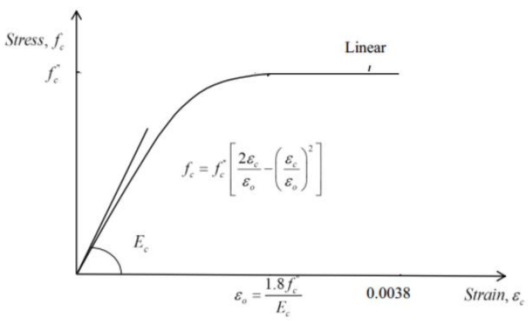

Figure 3. Concrete constitutive model.

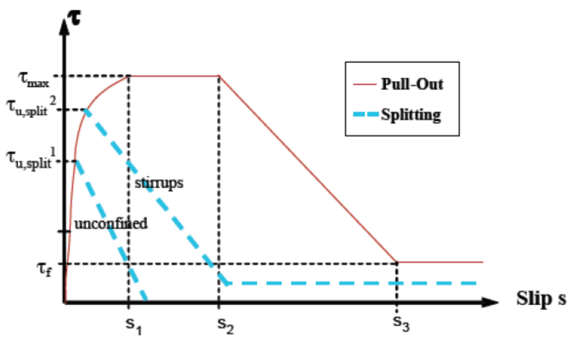

Figure 4. Bond slip law [6].

Steel - Both transvers and longitudinal reinforcement were modeled using LINK180 elements. The behavior of steel can be described by a perfect-elasto model, a linear increase segment with stiffness E_s and ends up with a horizontal segment when it reaches ultimate stress. Hence, a large deformation should be observed when the reinforcement yield. Besides, the reinforcement elements followed same constitutive law in both compression and tension.

Bond - COMBIN39 element was used in this paper to model the bond behavior between reinforcement and concrete. It is capable of being dimensionless and incorporating nonlinear generalized force-deflection. The transvers and top steel elements were assumed to be fully bonded with concrete. Bottom reinforcement was assumed to be bonded with concrete in transverse direction. The nodes of concrete and the nodes of reinforcement at the same coordinate were connected along longitudinal direction through COMBIN39 to simulate the bond behavior of RC members. The bond behavior was modeled according to CEB-FIP Model [6] shown in Fig.4.

Nodes of concrete element SOLID65 and nodes of steel elements LINK180 at the same locations were connected through dimensionless elements COMBIN39 in longitudinal direction. And they are fully bonded in transvers direction.

Verification against the experimental data

The meshing finite element model could affect the numerical analysis result. Before the analytical model were tested and investigated, the effect of element meshes have to be studied firstly to guarantee the accuracy of the finite element model. So three types of smeared element meshes (details listed in Table 1.) were used in meshing study here, half beam coarse mesh, half beam fine mesh, and full beam fine mesh.

Table 1. Details of different mesh

	Node number	Element number	Bond between concrete and longitudinal reinforcement
half beam coarse model	754	605	Fully Bonded
half beam fine model	2115	1024	Via spring elements
Full beam fine model	21385	18440	Fully bonded

The load-deflection responses of these beams are listed in Fig 5. Although the full beam fine model which consists of tremendous nodes and elements, the response of the beams are almost the same. And their ultimate loads were all close to the experiment results. The stiffness of FEM solution is higher than the real beam because there are some the initial micro cracks in experiment beams, which the FEM beams don’t have. Since their flexural strength all match the experiment ones well, it is unnecessary to use the full beam model with fine mesh to achieve little improvement in accuracy at the expense of tremendous computing time.

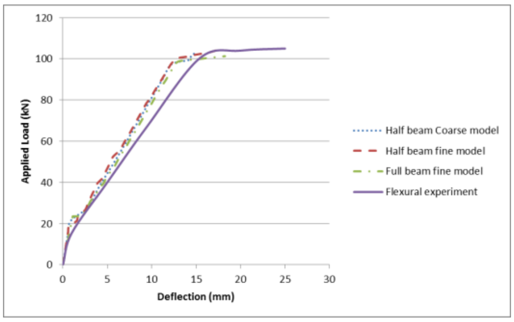

Figure 5. FEM results against flexural experiment.

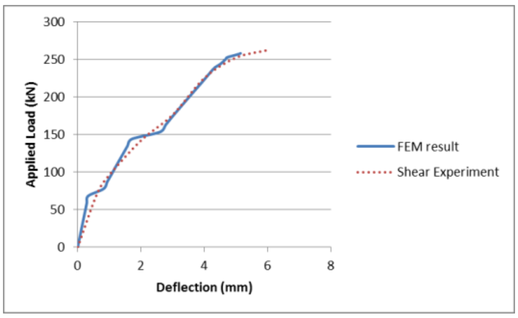

Figure 6. FEM result against shear experiment.

Shear experiment - Another finite element model has been set up to simulate the BN50 beam fail in shear with splitting cracks. The solution of the finite element mode is presented in Fig 6. As can be seen, the load deflection response of FEM also matches the shear failure beam with weak bond very well.

The element mesh of half beam fine model, shown in Fig 7 was chose to study the flexural strength of RC beams in later analysis due to the computer hardware performance and analysis time limit. To reduce the load concentration at point where the loads are applied, a high stiffness steel cushion was added to distribute the load evenly. At the left end near the support, an additional high stiffness steel cushion was also added to avoid concentration here. In order to investigate the effects of localized low strength concrete, 30 cases of the half finite element models with different inputs were built in ANSYS.

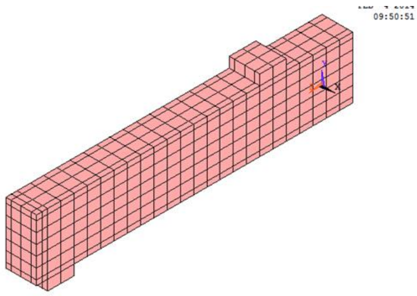

Figure 7. Finite element meshing.

Result analysis

Once we achieved the ultimate load of FEM solutions, we need to carefully investigate the failures modes of each case. In general, there are three types of failure modes:

Flexural failure: Generally, there are two types of flexural failures. The flexural failures controlled by tension (**F-T**), whose tension reinforcement yields firstly and generate large deformation that will damage the concrete eventually. This is an expected failure mode because it's ductile failure. And yielding of reinforcement exhibits a leveling branch in the load deflection response of structure. The second one is flexural failure controlled by compression (**F-C**), i.e., the concrete reaches its critical status and crushes before the reinforcement develop its full strength. This is an unexpected failure mode because the flexural member fails in a sudden.

Bond failure (**B**): In a flexural failure or a pure shear failure, before the reinforcement yield and concrete crush, the reinforcement nodes right above the support should only experience a small amount of movement during the loading. After the reinforcement yield or concrete crush, the reinforcements resist the deformation and the nodes above the support may start to move dramatically. However, in a bond failure, when the load increases, the concrete provide bond resistance and prevent the reinforcement from movement. When the load increase to a certain amount, before the reinforcement yield and less than the concrete critical point, the bond force in reinforcement are not enough to hold the reinforcement in its position, then the nodes right above the support will start to move dramatically. Soon the beam fails due to the relative movement between reinforcement and concrete. This situation is considered a bond failure.

The distinctive characteristic of a bond failure can be told from the bond stress/strain shape along the reinforcement. A normal flexural failure whose reinforcement could develop its stress sufficiently would look like a wave, which has several peaks and troughs. The relative movement between concrete and reinforcement at the end is extremely small. Because in real world, the high ribs of deformed bars are designed to create high mechanical bond force through interlock mechanism between concrete and reinforcement. However, for a beam which does not provide enough development length. The bond stress along the reinforcement will be a triangle or trapezoid shape, whose relative movement between reinforcement and concrete at the end are higher than other positions. The failure mechanism of each FEM cases was carefully investigated and then summarized in Table 1.

Table 1 Case failure summary

Case Name	Honeycombing Location	Strength of Concrete(MPa)	Ultimate Load (kN)	Failure Mode	Residual load capacity
16-0	None	42.7	104.21	F-T	1.000
16-1-30	1	30	101.23	F-T	0.971
16-1-20	1	20	87.76	F-C	0.842
16-1-10	1	10	53.62	F-C	0.515
16-2-30	2	30	102.17	F-T	0.980
16-2-20	2	20	95.97	F-C	0.921
16-2-10	2	10	60.26	F-C	0.578
16-3-30	3	30	103.82	F-T	0.996
16-3-20	3	20	103.81	F-T	0.996
16-3-10	3	10	72.27	F-C	0.693
25-0	None	42.7	178.92	B	1.000
25-1-30	1	30	161.03	B	0.900
25-1-20	1	20	95.18	F-C	0.532
25-1-10	1	10	60.94	F-C	0.341
25-2-30	2	30	167.37	B	0.935
25-2-20	2	20	115.15	F-C	0.644
25-2-10	2	10	67.93	F-C	0.380
25-3-30	3	30	173.98	B	0.972
25-3-20	3	20	128.90	B	0.720
25-3-10	3	10	101.57	F-C	0.568
32-0	None	42.7	195.42	B	1.000
32-1-30	1	30	161.32	B	0.825
32-1-20	1	20	95.63	F-C	0.489
32-1-10	1	10	66.05	F-C	0.338
32-2-30	2	30	173.45	B	0.888
32-2-20	2	20	152.48	B	0.780
32-2-10	2	10	79.69	F-C	0.408
32-3-30	3	30	183.56	B	0.939
32-3-20	3	20	175.68	B	0.899
32-3-10	3	10	101.98	B	0.522

*For the failure mode, B indicates bond failure, F-C indicates flexural failure that concrete crush firstly, F-T indicates that flexural failure that reinforcement yield firstly. Residual load capacity is calculated from the ultimate load of each case divided by the ultimate load capacity of cases with good concrete, Case 16-0, 25-0, or 32-0.

Proposed Approach

Generalized approach

Relative Defective Concrete Position (x/L) - This parameter is defined the same as the one mentioned before. It is the ratio of distance from center of localized poor concrete to the support end. x is the distance of the center of honeycombing concrete to the support end. L is the span length of the beam

Localized concrete strength (f'c) – This is the localized low strength of the problem concrete, which can be obtained by experiment.

Generalized Structural Performance Index Number (GSPI) - Generalized structural performance index number is defined by Eq. 1.

$$GSPI = \frac{P_{localized}}{P_{ACI}} \tag{1}$$

where, $P_{localized}$ indicates the ultimate loads of beams with localized concrete problem, P_{ACI} indicates the original ultimate load of beams with good concrete. The original structural member is assumed to have sufficient development length that the reinforcement and yield before the concrete crush. In this case, the predicted ultimate load PACI can be computed based on nominal moment from ACI equation.

Development Sufficiency (L_{pd}/L'_d) - This is a parameter introduced to reveal the development sufficiency of the reinforcement. It is related to rebar sizes and span length and concrete strength. L_{pd} is provided development length, for beams subjected to concentrated load; the provided length is from the support to where the applied load is. For beams subjected to uniform load, the provided development length is half the beam. L'_d is modified localized development length after modification of the development equations in ACI318M [7] in order to consider beams which subjected to localized concrete degradation using superposition. It can be described as Eq. 2, 3.

For the No. 19 and smaller deformed bars,

$$L'_d = \sum_i^n \Phi_i \frac{L_i}{L} \times \left(\frac{f_y}{2.1\sqrt{f'_{ci}}}\right) \times d_b \tag{2}$$

For the No. 22 and bigger deformed bars,

$$L'_d = \sum_i^n \Phi_i \frac{L_i}{L} \times \left(\frac{f_y}{1.7\sqrt{f'_{ci}}}\right) \times d_b \tag{3}$$

where, Φ is location related factor, for concrete defect occurs in bond sensitive region, Φ=1.2 {from 0(x/L) to 0.167(x/L)}; for concrete defect occurs in shear sensitive region, Φ=1.05{from 0.133(x/L) to 0.333(x/L)}; for concrete defect occurs in bending sensitive region Φ=1{from 0.333(x/L) to 0.5(x/L)}; L_i is the length of i th segment of beam; f'_{ci} is the localized strength for i th concrete; L is the total span length; d_b= the diameter of the tensile reinforcement; f_y is the yielding strength for the tensile reinforcement.

All 30 sample data calculated and discussed in Table. 2 were processed through generalized approach. Then these existing samples (As shown in Fig.8) plus some boundary condition data were used to generate an expanded generalized practical database for RC members subjected to localized low strength concrete. A modification of Shepard’s method was used here to do the 3-D interpolation job. The algorithm is described as a smooth function Q(x,y,z) (as shown in Eq. 4) which interpolates a set of m scattered data points (x_r,y_r,z_r,f_r) for r=1,2,…m, using a modification of Shepard's method, and then evaluates the interpolant at the set of selected points (u_r,v_r,w_r), as well as its first partial derivatives. The surface is assumed to be continuous and has continuous first partial derivatives.

$$Q(x,y,z,)=\frac{\sum w_r(x,y,z)\times q_r}{\sum w_r(x,y,z)} \tag{4}$$

where $q_r=f_r w_r(x,y,z,)=(1/d^2_r)$, and $d^2_r=(x-x_r)^2+(y-y_r)^2+(z-z_r)^2$. x,y,z were substituted with the predefined three variables, relative defective concrete position, development sufficiency, and localized concrete strength. After the 3-D interpolation is done, a generalized practical database produced from existing data can be set up. This empirical model can be visually presented in Fig.9. The bubble size and color are scaled according to the global structural performance index number after linear interpolation. Slices of the generalized practical model are listed in later figures.

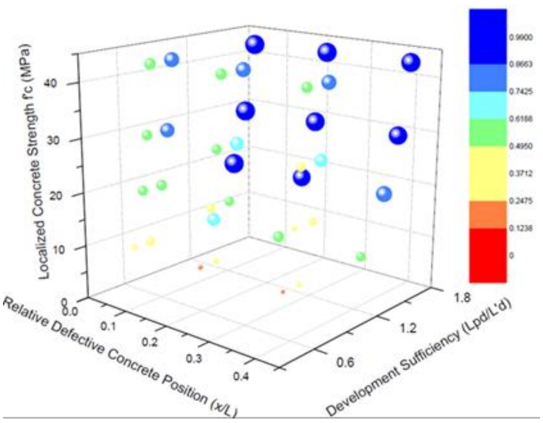

Figure 8. Existing sampling data.

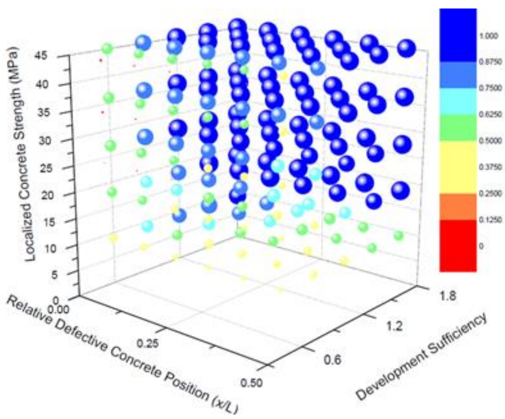

Figure 9. Generalized practical model.

Discussion

In terms of the effects of localized locations, in general, the middle part of concrete is the most critical position in honeycombing problem. Figs. 10, 11, and 12 were extracted from the generalized practical model when relative defective concrete position equals to 0.09, 0.27, and 0.41. When the development length is sufficient (>1.5), location has little impact on the structural performance. The structural performances are almost the same. When the development length is insufficient (=0.6), the middle part of the beam is the most critical location. It could at most reach as low as 0.35 performance index, but the support region can reach about 0.55 performance value. When the construction location move to middle, the acceptable performance area (GSPI>0.55) become smaller.

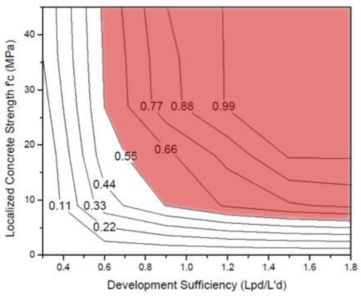

Figure 10. GSPI (x/L)=0.09.

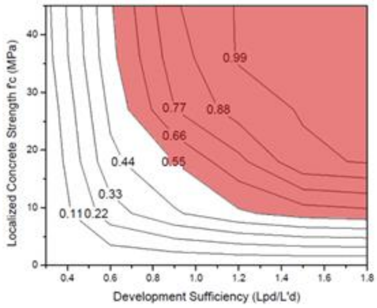

Figure 1 GSPI (x/L)=0.27

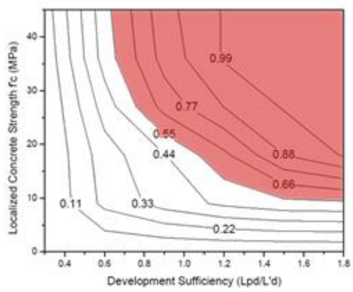

Figure 2 GSPI (x/L)=0.41

Development sufficiency plays a vital role to secure the load capacity of structural members. Large rebar require longer development length, and end up with a smaller red area. Figs. 13, 14, and 15 were extracted from the generalized practical model when development sufficiency equals to 0.6, 1.2, and 1.8. When the development is extreme insufficient, increase the localized concrete strength will contribute very little improvement on the structural capacity. And the beam may not be able to take full advantage of its material strength. When the development sufficiency increase, the acceptable performance area (GSPI>0.45) become bigger.

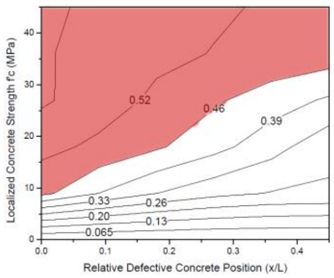

Figure 3 GSPI (L_{pd}/L'_{d})=0.6

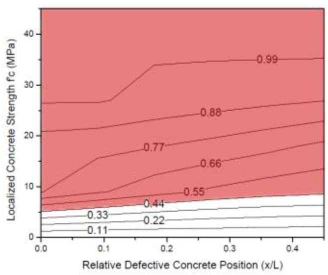

Figure 4 GSPI (L_{pd}/L'_{d})=1.2

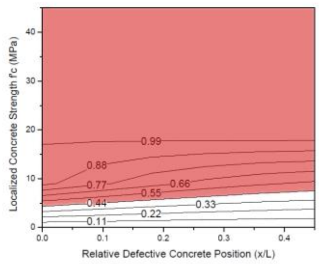

Figure 5 GSPI (L_{pd}/L'_{d})=1.8

Figs. 16, 17, and 18 were extracted from the generalized practical database when localized concrete strength equals to 9MPa, 27MPa, and 45MPa. Localized concrete strength could affect the structural performance of RC beams a lot. When the localized concrete strength is high enough, wherever the honeycombing occurs, there is little effect on the load capacity. When the concrete strength is low, the middle of the beam is the most critical place. When the localized concrete strength (f'_c) gets bigger, the acceptable performance area (GSPI>0.55) become bigger.

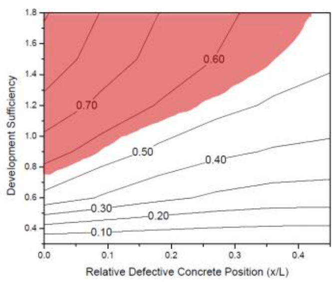

Figure 16. GSPI(f'c=9MPa)

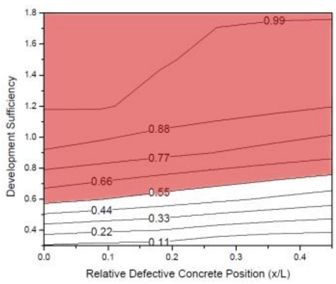

Figure 17. GSPI(f'c=27MPa)

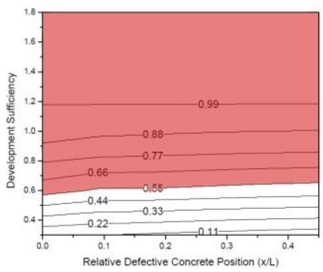

Figure 18. GSPI(f'c=45MPa)

Practical Analysis Procedure

With the generalized empirical model abstracted from finite element experiment, it is feasible to assess the structural performance for a flexural reinforced concrete member with localized low strength concrete problem by following these steps: (1) Measure the distance from the center of the void concrete to the beam's support end. (x/L). (2) Rebound hammer test can be used to inspect the localized concrete strength f'c. (3) Get the reinforcement information and calculate the development sufficiency (L_{pd}/L'_d). (4.a) Using the structural performance chart provided in the paper from Figs 10, 11, 12, 13, 14, 15, 16, 17, and 18. And do a linear interpolation from these generalized structural performance charts to get the GSPI value for the inspected members. (4.b) Or constructing own generalized structural performance index charts with respect to the structural member for the localized concrete problem based on experiment and the generalized approach discussed before. (5) Read the structural performance index from the chart, and calculate the remaining flexural capacity by multiplying the structural performance number and the original flexural performance, which is assumed to be the nominal flexural moment according to ACI code.

Conclusions

An approximate structural performance assessment approach was proposed in the paper and can be used to predict the performance of RC beams considering the localized low strength concrete problems .Although the approach is proposed from the concentrated load, it could also work for the distributed load in order to get the nominal moment. When using the GSPI chart provided in the paper, to get a better accuracy, the range of the input parameters should fall within the existing FEM sample data.

Both the FEM results and proposed structural performance charts show the impact of localized concrete problem. In order to have the flexural structural members perform as design: (1) Make sure that the development length is enough. Large rebar size and short span may cause development insufficiency and the beam may fail in bond. (2) Make sure that the localized strength of concrete wouldn't affect the load capacity of the structural member too much to keep it safe. (3) Make sure that the location where the construction defect occurs would not damage the load capacity of the beam a lot.

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