INTRODUCTION

Ultra high strength concrete (UHSC) is a new construction material. It is investigated and applied in developed countries during several recent decades. Key properties of UHSC are ultra high strengths, from 100 to 200 MPa in compression and more than 40 MPa in flexural strength, shear strength improved, high resistances in impact as well as repeated loads. Especially, UHSC also maintains high durability and long-term stability. This material has been investigated and applied in bridges, high rise buildings and other special constructions to enhance load bearing as well as durability of the structures.

In Viet Nam, infrastructures have been developed. Modern bridges and highways have been building. Consequently, it is necessary to research and develop a new concrete with ultra high strengths and durability.

It is allowed to investigate and apply Ultra high strength concrete (UHSC) manufactured by using domestic compositions. The UHSC will be used for the modern construction structures to replace for traditional bridges and highways.

In according to the above reasons, the author designed to investigate this thesis: "Investigation in compositions, mechanical properties of ultra high strength and its application in bridge structure".

Objectives:

In theory: gradation theory to obtain an optimum density in accordance of Larard's theory. Guidelines to calculate optimum gradations in accordance of Fuller in 1997. Experimental investigations determine proportions in accordance of SETRA/AFGC in 2002; selecting proportion in accordance of DIN; selecting proportion in accordance of ACI-544. These references were used in this investigation thesis.

Experimental research: modify and correct proportions by experiments and from the experiments to adjust coefficients of the formulas of concrete proportions. This is also a methodology used in South Korea and America. Methodology and objective of this investigation are to correct the modeling of material compositions in Viet Nam after running experiments and also using results from the experiments to adjust a bending strength formula used for structural analysis.

Objective: Using domestic materials to run experimental investigations and determine modeling of material, and then manufacture UHSC, from 120 to 140 MPa, as well as to apply it in structures.

Scope of investigation: Correct the modeling of material via experiments, experimental analysis the bending behaviour of beams to determine σ_t, experimental analysis the bending behaviour of beams to determine their new height. The thesis investigates experimental beams under static loads only, dynamic and repeated loads have not carried out.

Scientific and realised values:

- ***In theory***: Research in application of theoretical calculations of optimum density to design proportion of UHSC. Analyse bending behaviour of beams and bridge beams to determine flexural strength σ_t and height of the beams.
- ***In experiments***: surveying materials, selecting proportions of UHSC, from 120-140 MPa using domestic materials. Basing on experimental results to propose mechanical properties of the UHSC as well as flexural strength σ_t; analyse bending behaviour of bridge beams to determine and their heights.

Chapter 1: REVIEW OF RESEARCHES AND APPLICATIONS OF UHSC OVER THE WORLD AND IN VIETNAM

1.1. References

UHSC is a new material that has been developed since 1990. Mechanical behaviours, formulas to select proportions as well as guidelines for designing and construction reported in France, America and Germany. Several first applications in Canada, Euro, Asia and America confirmed advantages of this new material in cost, durability and other properties.

Excellent properties of the UHSC allow to think of manufacture UHSC using domestic materials basing on references of investigated results published over the world. This opens a new trend for construction materials and structures.

1.2. Investigated UHSC in America, Euro and Asia

New theories of gradation in according to optimum density presented by Larard;

Theories of optimum gradation presented by SETRA/AFGC;

Guidelines for design and construction investigated and proposed by RILEM, DIN;

Experiments to correct modeling of material carried out by FHWA (America) and South Korea.

Figures from 1.1 to 1.6 introduce bridge, building structures and military applications.

Fig 1.1. Comparison in weight and height of beams casted from UHSC and traditional concretes.

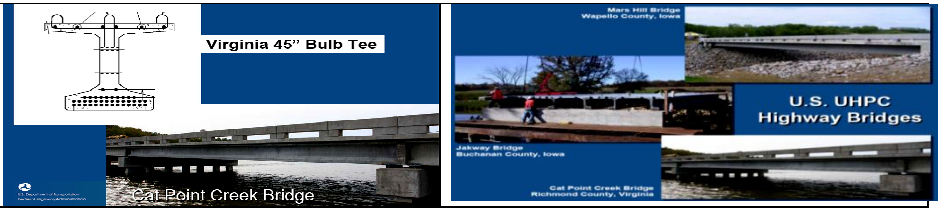


Fig 1.2. Bridges used UHSC to cast T and Π beams in America

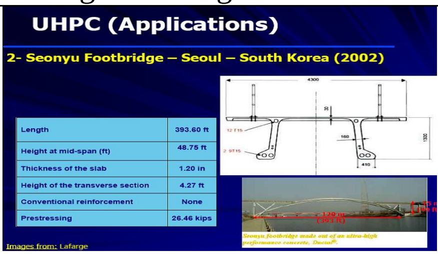


Fig 1.3: Footbridge in Seoul, South Korea, 2002.

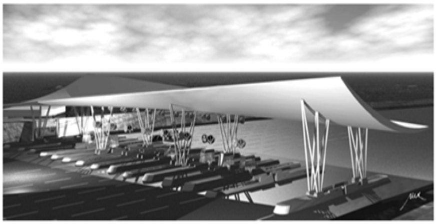


Fig 1.4: Milau roof, 2004.

Fig 1.5: Bourg –lès – Valence Bridge, France, 2004

Fig 1.6: Explosive test in Iran Military

1.3. Relevant researches published in Viet Nam

In Viet Nam: UHSC is a relative new subject. In 2008, several researchers at the University of Transportation and Communication, University of Construction, Ho Chi Minh City University of Polytechnics started to investigate this concrete. The investigation from those Institutions are initial researches in UHSC in Viet Nam.

The UHSC is a hot subject in over the world and also in Viet Nam. It is necessary to pay attentions in research and manufacture UHSC using domestic materials to contribute understanding of fundamental, designing and application of this material in construction.

1.4. Objective

Using domestic materials and basing on guidelines to investigate and manufacture UHSC, from 120 to 140 MPa. Experimental research in bending of reinforced concrete beams casted by UHSC to determine K coefficient in formula of flexural strength. Analyse bending behaviour of the bridge beams using UHSC to propose height of the beams.

1.5. Content and methodology

Select materials, design proportion, test mechanical properties of UHSC, from 120 to 140 MPa. Analyse bending of beams, bridge beams and propose the use of UHSC in structures. Using theories and experiments to determine proportions, mechanical properties of the UHSC and formula of flexural strength as well as height of bridge beams.

Chapter 2: MATERIALS AND DESIGN OF PROPORTION OF UHSC

2.1. Materials

2.1.1. Cement, superplasticiser and silica fume

This investigation used PC40 But Son cement, grade 1, agreed with international grade and the use of Viet Nam.

Superplasticiser is a Policacbol silat supplied from Sika Viet Nam, label 3000-20, properties of the Superplasticiser agrees with ASTM C494, group C. Silica fume was supplied also by Sika Viet Nam. The properties of this additive agree with ASTM 1230-95a, Figure 2.1.

Fig 2.1. Silica fume

2.1.2. Coarse aggregate and quartz powder

Coarse aggregate: using quartz sand agreed international guidelines. The quartz sand was ground from quartz rock that exploited at Thanh Son-Phu Tho. The author prepared the quartz sand (as coarse aggregate in the gradation of the UHSC) with maximum size of 0.6 mm, gradation as presented in Table 2.1 and Figure 2.2.

Table 2.1. Gradation of quartz sand

Size (mm)	Passing, A%
0,63	100
0,315	67,1
0,14	41,6
0,075	13,9

Quartz powder was also ground from quartz rock Thanh Son-Phu Tho with particle size of approximately 27.9μm as in Figure 2.3.

Fir 2.2: Quartz sand

Fig 2.3: Quartz powder

2.1.3. Steel fibre

Using Dramix steel fibre from BeKeart, Germany, grade OL13-20, diameter of 0.2 mm, length of L=13 mm. Yield strength is 2000 MPa, content of fibre is 2% by volume, as Figure 2.4.

Fig 2.4: Steel fibre

In short, main materials prepared to mix UHSC are PC40 But Son cement, quartz sand and quartz powder ground from quartz rock of Thanh Son – Phu

Tho, silica fume and superplasticiser supplied from Sika Viet Nam, Dramix steel fibre imported from ShangHai, China. It was shown that there are enough resources of materials in Viet Nam agreed with international standards to manufacture UHSC.

2.2. Manufacture UHSC in accordance of theory of the optimum density

2.2.1. Introduction

In this thesis, theory of the optimum density of Mooney and Larrad was used to investigate, the optimum gradation curve of Fuller was used as a comparison.

2.2.2. Selection proportion

Base on the optimum density of Mooney, researches of Thomson and Larrard, the author carried out calculation and set up three formulas of UHSC as C1, C2 and C3 in Table 2.2.

Table 2.2: Proportions of UHSC

Materials	C1	C2	C3
But Son PC40 cement, kg/m^3	800	850	900
Silica fume (25%X), kg/m^3	195,5	195,5	207
Quartz sand Q1, kg/m^3	900	935	977
Quartz powder Q2, kg/m^3	280	150	120
Steel fibre, kg/m^3	160	170	160
Superplasticiser, kg	16	17	18
Water, lít	160	170	170
N/X ratio	0,20	0,20	0,20

Gradation with maximum size of 0.6 mm, minimum size is 0.00001 mm as in Figure 2.5.

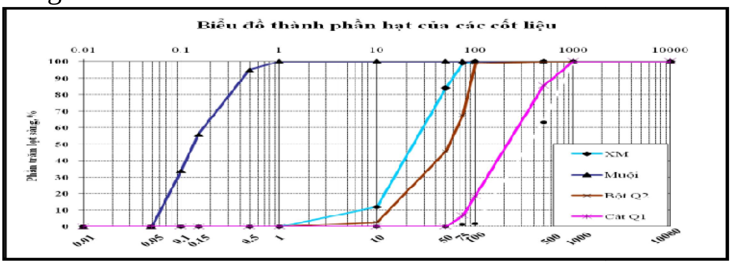


Fig 2.5: Gradation of UHSC

2.2.3. Gradation check

Base on concrete formulas, create gradation of UHSC and compare to the optimum gradation in according of Fuller as in Figure 2.6.

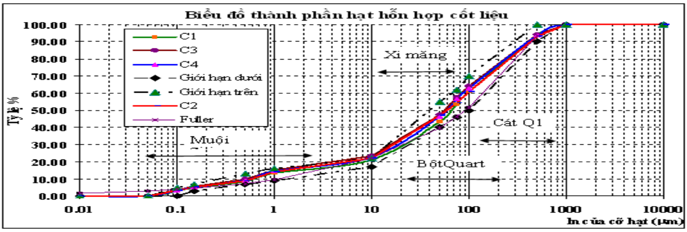


Fig 2.6: Gradation of UHSC in comparison with the Fuller gradation

Tested results showed that designed gradations C1, C2 and C3 are very close to Fuller's gradations.

Results obtained in Chapter 2 includes:

- Extract and ground quartz sand and powder agreed with standards.
- Selected cement, silica fume, steel fibre agreed with UHSC.
- Using a model of the optimum density to design proportions of UHSC C1, C2 and C3.
- Tested gradations that agreed with France researches and Fuller's optimum gradation.

Chapter 3: TESTS OF COMPRESSIVE STRENGTH, BENDING STRENGTH AND ELASTIC MODULUS OF UHSC

3.1. Introduction

In this Chapter the author presents tests of compressive strength, specific tensile strength and elastic modulus of UHSC.

3.1.1. Compressive strength

Compressive strength was determined at the ages of 3, 7 and 28 days. Samples were cylinders with dimensions of 10×20 cm (diameter × height). The samples were cured in room condition.

3.1.2. Flexural strength

Bending behaviour of materials was characterised by three tests as below:

- Tensile strength in elastic bending of UHSC (f_{tj}). This tested value was determined proportionally with elastic deformation at the time of a first crack with a relative deformation of 1‰, opening crack width of 0.05 mm and a deflection of less than 1 mm.

- Normal maximum flexural strength (due to maximum bending moment) with a deformation of 3‰.

- Flexural strength at a time of maximum deformation with a deflection of tested beam of 10 mm. Bending were tested in accordance with European standards (RILEM).

3.1.3. Procedure to test the samples and analyse

Two tests proposed in the world:

Type 1: Four point bending test applied for prism samples without notch that allows to find out tensile strength after adjusting several proportional coefficients.

Type 2: Three point bending test applied for prism samples with notch, using back-calculation method as guideline of RILEM.

The author used four point bending test applied for beams in accordance of European guideline (Figure 3.1).

3.1.4. Dimensions of samples (European standards)

The prism samples with cross section in square (a=15 cm) and length of 4a (60 cm).

a. Test equipments

The four point bending test in accordance of European guideline specifies that measurement equipment must be fixed on the samples to measure real deflections of the samples (Figure 3.1).

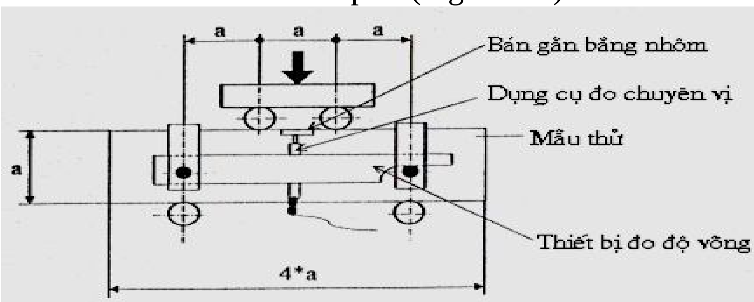


Fig 3.1: Mode of four point bending test

b. Testing result collection

Tested figures carry out with a frequency of 5 Hz. They are:

+ Deflection
+ Load
+ Load-deflection diagram.

c. Calculation of opening crack width and deformation

Given deflection f_0 with the last stage of elastic, opening crack width (w) was analysed via a relation with deflection in accordance qith SETRA-AFGC.

3.2. Sample preparation

3.3. Tested results:

Results of flow test, compressive strength are presented in Tables 3.1; 3.2; 3.3 and Figures 3.2; 3.3.

Table 3.1: Flow test results

Sample	C1	C2	C3
Slump (cm)	24,00	29,00	27,00
Flow (cm)	45,00	64,00	50,50
Date of cast	29/3/2011	1/4/2011	6/4/2011

Fig 3.2: Trial mix

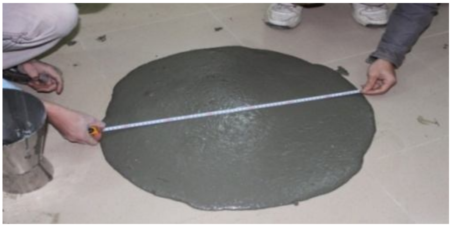


Fig 3.3: Flow test

Table 3.2: Compressive strength test

No	Label	Date of cast	Compressive strength (MPa)								
			R3	TB3	S3	R7	TB7	S7	R 28	TB28	S28
C1	C11	29/3	65,89	69,77	3,32	109,89	106,59	5,33	134,70	127,59	5,22
	C12	29/3	66,53			100,63			122,63		
	C13	29/3	71,72			101,23			126,90		
	C14	29/3	74,65			111,76			132,63		
	C15	29/3	72,48			102,36			119,79		
	C16	29/3	67,36			113,69			128,90		
C2	C21	1/4	68,55	72,65	3,69	111,47	112,46	5,28	121,36	130,01	5,73
	C22	1/4	67,89			106,34			128,63		
	C23	1/4	71,66			115,19			137,24		
	C24	1/4	75,12			120,69			133,68		
	C25	1/4	78,34			115,31			124,36		
	C26	1/4	74,35			105,73			134,80		
C3	C31	6/4	82,42	84,75	5,07	115,51	113,06	5,57	142,56	139,21	6,21
	C32	6/4	80,23			112,36			132,21		

	C33	6/4	77,64			105,61			129,38		
	C34	6/4	86,62			122,38			144,77		
	C35	6/4	91,65			107,34			145,61		
	C36	6/4	89,92			115,18			140,74		

R_i: Compressive strength at the day i

TB_i: average compressive strength at day i

S_i: standard deviation of compressive strength at day i

Table 3.3: Average compressive strength of sets of samples

Set	Average compressive strength (MPa)	Standard deviation (S)	Relative deformation (‰)
C1	127,59	5,22	4,02
C2	130,01	5,73	3,55
C3	139,21	5,21	3,75

From compressive strength tests of three mixtures C1, C2, C3, drawing graphs of relationships between strength-time and strength-water/binder ratio as in Figures 3.4 and 3.5.

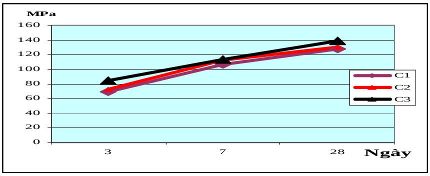


Fig 3.4: Compressive strength Vs time

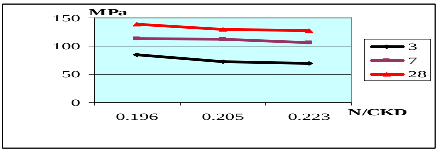


Fig 3.5: Compressive strength Vs water/binder ratio of C3 mix

+ Flexural strength tested result

Four point bending test was carried out at the University of Transportation and Communications. Procedure was accordance of RILEM as in Figure 3.6.

Fig 3.6: Bending test and damaged mode

Tested results are presented in Table 3.4 and Figure 3.7

Table 3.4: Relationship between load and deflection

Deflection δ (mm)	Load P (kN)					
	P_{M1}	P_{M2}	P_{M3}	P_{M4}	P_{M5}	P_{M6}
0,00	0,000	0,000	0,000	0,000	0,000	0,000
0,20	75,470	70,637	112,226	80,176	73,181	97,091
0,22	80,303	78,777	118,204	94,421	76,361	101,161
0,25	83,865	82,974	126,598	107,775	80,558	106,884
0,30	94,039	100,653	142,750	148,219	90,351	119,475
0,40	107,520	119,094	162,209	207,995	106,249	126,343
0,50	112,862	122,910	179,124	227,199	118,077	128,251
0,70	115,152	123,673	205,196	247,930	126,216	132,066
1,00	119,094	123,673	210,284	291,554	126,343	132,066
2,00	89,969	79,413	159,792	219,000	90,732	78,014
3,00	66,949	57,029	103,959	143,667	73,181	59,446
5,00	29,939	32,864	57,029	106,000	51,051	29,558
10,00	12,134	11,116	8,191	42,420	22,817	9,336

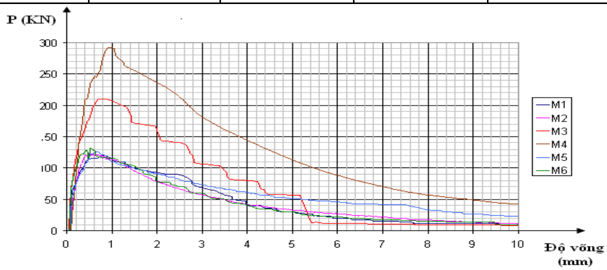


Fig 3.7: Graph of load and deflection

A relationship between strength and opening crack width, strain … in case of four point bending test is calculated in accordance of SETRA/AFGC, results as in Table 3.5.

Table 3.5: Relation between strength and deformation of UHSC

Sample	Deflection (mm)	Opening crack width W (mm)	Deflection ($^{o}/_{oo}$)	Load P(kN)	Flexural strength Ru (MPa)	Specified strength 0,7265xRu (MPa)
C1	0,092	0,05	0,2	73,47	9,80	7,12
	0,2	0,18	2	79,50	10,60	7,70
	0,3	0,30	3	122,68	16,36	11,88

	0,9	1,02	10	97,74	13,03	9,47
	2,12	2,48	25	84,17	11,22	8,15
	2,55	3,00	32	0,00	0,00	0,00
C2	0,092	0,05	0,2	85,05	11,34	8,24
	0,2	0,18	2	88,51	11,80	8,57
	0,3	0,30	3	129,20	17,23	12,52
	0,9	1,02	10	110,42	14,72	10,70
	2,12	2,48	25	84,23	11,23	8,16
	2,55	3,00	32	0,00	0,00	0,00
C3	0,092	0,05	0,2	90,47	12,06	8,76
	0,2	0,18	2	126,26	16,83	12,23
	0,3	0,30	3	251,19	33,49	24,33
	0,9	1,02	10	210,67	28,09	20,41
	2,12	2,48	25	159,74	21,30	15,47

+ Stress-strain model

Drawing a graph of stress-strain in accordance of SETRA/AFGC for sets of C3 samples as a fundamental for structural analyse, Figure 3.8.

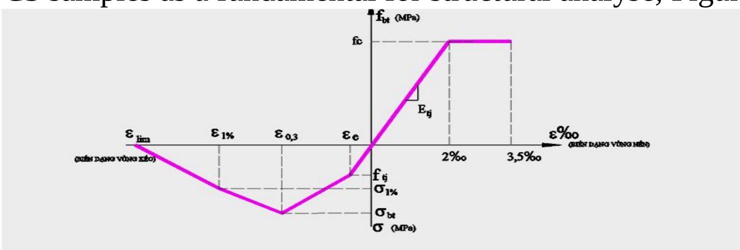


Fig 3.8: Graph of stress – strain of UHSC, samples C3 drawn as SETRA/AFGC

+ Elastic modulus test

- Elastic modulus and poison coefficient tests of UHSC carried out as ASTM, cylinders with diameter of 15 cm and height of 30 cm. Testing equipment is a 150 tons (1500 kN) machine, as Figure 3.9.

Fig 3.9: Elastic modulus test

Average tested results are presented in Table 3.6.

Table 3.6: Elastic modulus tested result

Set of samples	**C1**	**C2**	**C3**
Compressive strength (MPa)	127,59	130,01	139,21
E (MPa)	46500	47200	49300
E= 9200 x $f_{cj}^{1/3}$	46085	46449	47565
Error	1,009	1,016	1,038

+Comments

It is shown from the results: E= 9200 x $f_{cj}^{1/3}$

Coefficient of K_0 =9200, between the range of European standards.

+Conclusion of compressive strength, flexural strength and elastic modulus of UHSC

Three trial mixtures showed that mix C3 (as in Table 3.7) obtained a maximum strength of 139,2 MPa, specified flexural strength of 24,22 MPa.

Table 3.7: Proportion of mix C3

Water, kg (final)	217,57 kg
Cement	900 kg
Quartz sand d=0,6mm (dry)	910 kg
Quartz powder d=27μm (dry)	120 kg
Silica fume d=1μm	207 kg
Steel fibre d=0,2mm	160 kg
Superplasticiser	22,46kg

3.4. Comments

The use of domestic materials prepared successfully UHSC with typical properties as below:

- Flow of fresh mix from 45 to 64 cm, agreed with international requirements of more than 50 cm.
- Compressive strength from 125,6 to 139,2 MPa at 28 days, relative deformation of approximately 3,5‰.
- Flexural strength at the time of first crack from 9,8 to 12,06 MPa, Maximum flexural strength from 16,36 to 33,49 MPa. Flexural strength at deflection of 10 mm from 2,03 to 3,9 MPa. Specified elastic strength from 7,12 to 8,76 MPa. Maximum specified strength from 11,8 to 24,22 MPa.
- Elastic modulus: 46,2-49,3 GPa. This value in a range of 45-55 GPa as investigations published.
- Stress-strain model used for calculation drawn as guidelines of Europe for C3 samples (Figure 3.8).

Chapter 4: EXPERIMENT INVESTIGATION AND ANALYSE BENDING BEHAVIOUR OF REINFORCED CONCRETE BEAM AND BRIDGE BEAM CASTED BY UHSC

4.1. Introduction

Investigated results of ACI-544 describes that flexural strength of fibre concrete is approximately 40 MPa grade.

Results from Imam et al (1995) calculated that flexural strength of high performance fibre concrete is less than 100 MPa.

Consequently, UHSC beams with compressive strength from 120 to 140 MPa should consider formulas for flexural strength. This investigation aims to analyse and find out a suitable formula for flexural strength (σ_τ) of UHSC based on experiments and theory calculations.

4.2. Fundamental for analyse flexural behaviour of reinforced UHSC beam

Using method from ACI-544 and Imam et al. (1995) (stress-strain graph drawn as in accordance of ACI-544 and Imam as in Figure 4.1).

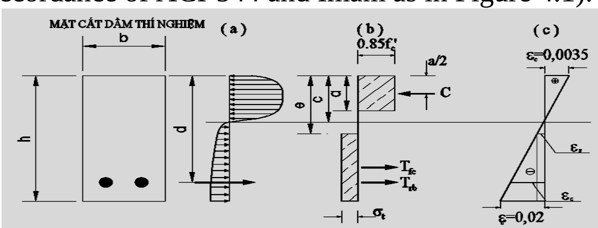


Fig 4.1: Graph of flexure of beams as ACI-544

(a): Load distribution; (b): Stress graph; (c): Strain graph

In accordance of ACI-544, formula to calculate bending moment of flexure beam using fibre concrete as Figure 4.1.

$$M_n = A_s.f_y.\left(d - \frac{a}{2}\right) + \sigma_t.b.(h - e).\left(\frac{a}{2} + \frac{e}{2} - \frac{a}{2}\right) \qquad (4\text{-}1)$$

where $\sigma_t = K.(l_f/d_f)\rho_f F_{be}$ (4-2)

σ_t: Flexural strength after cracking of steel fibre concrete

where:

+ In accordance of ACI, K=0,00772.

+ As Imam et al (1995), K=0,0138.

In short, steel fibre UHSC with strength more than 130 MPa needs to adjust a suitable K*, or other word, need to find out a suitable σ_t .

4.3. Prepare samples

In this section, use the mix C3 as describe in Chapter 2 and Chapter 3.

Cast 3 sets samples (9 beams) as accordance of ACI544 with width of 125 mm, height of 250 mm and length of 2400 mm.

Set 1: 3 beams, used 2 rebars of ∅12mm, label of 2D12-1; 2D12-2 and 2D12-3.

Set 2: 3 beams, used 2 rebars of ∅16mm, label of 2D16-1; 2D16-2 and 2D16-3.

Set 1: 3 beams, used 2 rebars of ∅20mm, label of 2D20-1; 2D20-2 and 2D20-3.

Samples and testing model as in Figures 4.2 and 4.3.

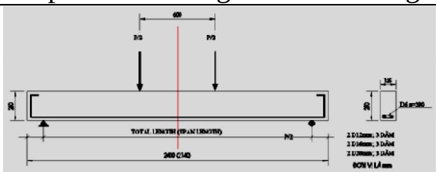


Fig 4.2: Structure and testing model of 9 beams

Fig 4.3: Samples ready for test

4.4. Method to test beam

Test was carried out at the University of Communications and Transportation (UCT). The author used four point bending test that agreed with European standards.

4.5. Tested results

From tested results of 9 beams (3 sets of samples) determined values of loads and deflection. Setting up graph of load-deflection (P - δ) as presented in Figure 4.4 and Table 4.1.

Table 4.1: Tested results of load-deflection relationship

BẢNG TỔNG HỢP SỐ LIỆU THÍ NGHIỆM TẢI TRỌNG ĐỘ VÕNG						
Ký hiệu dầm	Vết nứt đầu tiên		Tải trọng lớn nhất		Kết thúc thí nghiệm	
	Độ võng (δ)mm	Tải trọng (P) KN	Độ võng (δ)mm	Tải trọng (P) KN	Độ võng (δ)mm	Tải trọng (P) KN
2D12-1	0,7701	39,2885	8,5658	85,4500	25,1527	70,2259
2D12-2	0,7910	36,0597	9,7157	75,8652	25,5288	60,5839
2D12-3	0,8810	37,8762	8,5589	79,5520	25,6883	68,2350
Gía trị TB2D12	0,8140	37,7415	8,9468	80,2891	25,4566	66,3483
2D16-1	0,7951	39,2988	8,6136	110,2215	25,2254	100,4483
2D16-2	0,9205	37,7876	10,1390	120,6879	25,0255	109,8215
2D16-3	0,8161	36,5826	9,0266	100,7261	25,2684	89,5581
Gía trị TB2D16	0,8439	37,8897	9,2597	110,5452	25,1731	99,9426
2D20-1	0,9905	47,8046	9,5268	186,2210	25,0215	175,6214
2D20-2	1,1526	51,4102	10,0552	193,5216	25,4591	185,2568
2D20-3	1,0682	56,7824	11,5563	200,7589	25,3357	188,6783
Gía trị TB2D20	1,0704	51,9991	10,3794	193,5005	25,2721	183,1855

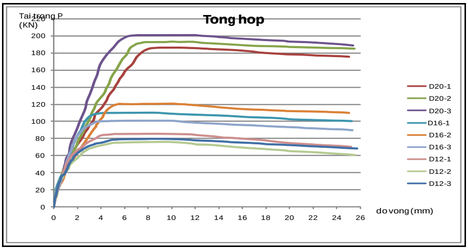


Fig 4.4: Relation of loads and deflections of tested beams

4.6. Comments

- Set 1 (beams used 2 rebars of ∅12mm) with the use of tensile rebars of 0.723% in ratio of cross section, load to create a first crack is P=37,741 kN in average, and deflection is δ=0,814mm in average; The average maximum load is P_{max}= 80,262 kN in proportional of deflection of δ=8,626mm; At the end of the test δ=25mm and average load is P=66,34 kN.
- Set 2 (beams used 2 rebars of ∅16mm) with the use of tensile rebars of 1.286% in ratio of cross section, load to create a first crack is P=37,889 kN in average, and deflection is δ=0,843mm in average; The average maximum load is P_{max}= 110.423 kN in proportional of deflection of δ=8,743mm; At the end of the test δ=25mm and average load is P=99,95 kN.
- Set 3 (beams used 2 rebars of ∅20mm) with the use of tensile rebars of 2.009% in ratio of cross section, load to create a first crack is P=51,999 kN in average, and deflection is δ=1,070mm in average; The average maximum load is P_{max}= 193,188 kN in proportional of deflection of δ=8,712mm; At the end of the test δ=25mm and average load is P=183,12 kN.
- As in the graph of load-deflection, before first crack occurred: Load-deflection relationship of the UHSC beams is similar as that of traditional reinforced concrete beams. However, after cracking the traditional beams occur a rapidly reduction in hardness and the cracks penetrated into compression area of the beam, this leads to suddenly collapsed. In case of

UHSC beams, the deflection continues to develop but slow, load is increased and then maintain horizontally, not sudden fall down. This could be due to energy is absorbed by steel fibre resulting in a further resistance of load and do not sudden collapsed.

Bending behaviour of the UHSC reinforced rebars in tensile area, after cracking, load continues to develop, tensile resistant ability and deflection develop and do not sudden collapsed. This demonstrates that the UHSC beams own a higher toughness. The relationship and values of loads, deflections are similar as results published in Germany and South Korea.

4.7. Calculate and analyse the experimental results

From deflection and load it is calculated w, M_{cr}, R_{ku}, ε_2 as guidelines of SETRA/AFGC, presented in Table 4.2.

Table 4.2: Calculated results of values at points of specified opening width cracks (CMOD)

XUẤT HIỆN VẾT NỨT ĐẦU TIÊN W=0						
TÊN DẦM TN	TẢI TRỌNG	ĐỘ VÕNG TN	Mcr(TN)	Rku (ĐT)	Mở rộng vết nứt	Biến dạng vùng kéo
	P (KN)	δ (mm)	KN.m	Mpa	W (mm)	ε2 (‰)
2D12-1	39,2885	0,7701	15,1261	11,6168	-	0,2582
2D12-2	36,0597	0,7910	13,8830	10,6621	-	0,2369
2D12-3	37,8762	0,8810	14,5823	11,1992	-	0,2489
TRUNG BÌNH 2D12	37,7415	0,8140	14,5305	11,1594	-	0,2480
2D16-1	39,2988	0,7951	15,1300	11,6199	-	0,2582
2D16-2	37,7876	0,9205	14,5482	11,1730	-	0,2483
2D16-3	36,5826	0,8161	14,0843	10,8167	-	0,2404
TRUNG BÌNH 2D16	37,8897	0,8439	14,5875	11,2032	-	0,2490
2D20-1	47,8046	0,9905	18,4048	14,1349	-	0,3141
2D20-2	51,4102	1,1526	19,7929	15,2010	-	0,3378
2D20-3	56,7824	1,0682	21,8612	16,7894	-	0,3731
TRUNG BÌNH 2D20	51,9991	1,0704	20,0196	15,3751	-	0,3417

GIAI ĐOẠN ĐỘ MỞ RỘNG VẾT NỨT W=0,3mm						
TÊN DẦM TN	TẢI TRỌNG	ĐỘ VÕNG TN	M(TN)	Rku (ĐT)	Mở rộng vết nứt	Biến dạng vùng kéo
	P (KN)	δ (mm)	KN.m	Mpa	W (mm)	ε2 (‰)
2D12-1	76,2118	2,9840	29,3415	22,5343	0,2952	2,2719
2D12-2	67,8655	2,9781	26,1282	20,0665	0,2916	2,1957
2D12-3	71,8795	3,1300	27,6736	21,2533	0,2999	2,2716
TRUNG BÌNH 2D12	71,9856	3,0307	27,7145	21,2847	0,2956	2,2464
2D16-1	105,6623	3,0142	40,6800	31,2422	0,2959	2,4696
2D16-2	87,2755	3,2057	33,6011	25,8056	0,3047	2,4017
2D16-3	95,4351	3,0188	36,7425	28,2183	0,2937	2,3893
TRUNG BÌNH 2D16	96,1243	3,0796	37,0079	28,4220	0,2981	2,4202
2D20-1	99,3816	3,2848	38,2619	29,3852	0,3059	2,4885
2D20-2	115,7677	3,4535	44,5706	34,2302	0,3068	2,6015
2D20-3	139,0131	3,2646	53,5200	41,1034	0,2929	2,6706
TRUNG BÌNH 2D20	118,0541	3,3343	45,4508	34,9062	0,3018	2,5869

GIAI ĐOẠN ĐỘ MỞ RỘNG VẾT NỨT W=0,5mm						
TÊN DẦM TN	TẢI TRỌNG	ĐỘ VÕNG TN	M(TN)	Rku (ĐT)	Mở rộng vết nứt	Biến dạng vùng kéo
	P (KN)	d (mm)	KN.m	Mpa	W (mm)	ε2 (‰)
2D12-1	84,4112	4,5603	32,4983	24,9587	0,5054	3,5869
2D12-2	73,2215	4,5210	28,1903	21,6501	0,4973	3,4653
2D12-3	77,4325	4,6782	29,8115	22,8952	0,5063	3,5467
TRUNG BÌNH 2D12	78,3551	4,5865	30,1667	23,1680	0,5030	3,5329
2D16-1	109,9689	4,6177	42,3380	32,5156	0,5097	3,7807
2D16-2	115,4441	4,7048	44,4460	34,1345	0,5046	3,7861
2D16-3	100,2568	4,6818	38,5989	29,6439	0,5154	3,7514
TRUNG BÌNH 2D16	108,5566	4,6681	41,7943	32,0980	0,5099	3,7728
2D20-1	133,1761	4,8216	51,2728	39,3775	0,5108	3,9400
2D20-2	155,8582	5,0537	60,0054	46,0842	0,5202	4,1451
2D20-3	186,7783	5,0871	71,9096	55,2266	0,5359	4,4425
TRUNG BÌNH 2D20	158,6042	4,9875	61,0626	46,8961	0,5223	4,1759

GIAI ĐOẠN ĐỘ MỞ RỘNG VẾT NỨT W=0,01H						
TÊN DẦM TN	TẢI TRỌNG P (KN)	ĐỘ VÕNG TN δ (mm)	M(TN) KN.m	Rku (ĐT) Mpa	Mở rộng vết nứt W (mm)	Biến dạng vùng kéo ε2 (‰)
2D12-1	74,4566	20,1256	28,6658	22,0153	2,5807	15,9742
2D12-2	66,0389	19,5125	25,4250	19,5264	2,4962	15,4117
2D12-3	72,4322	19,6956	27,8864	21,4168	2,5086	15,5282
TRUNG BÌNH 2D12	70,9759	19,7779	27,3257	20,9862	2,5285	15,6381
2D16-1	103,2603	19,5351	39,7552	30,5320	2,4987	15,6711
2D16-2	111,9945	19,7566	43,1179	33,1145	2,5115	15,8053
2D16-3	93,4385	19,5255	35,9738	27,6279	2,4946	15,5821
TRUNG BÌNH 2D16	102,8978	19,6057	39,6156	30,4248	2,5016	15,6862
2D20-1	178,4016	19,7255	68,6846	52,7498	2,4980	16,1608
2D20-2	187,2772	19,9522	72,1017	55,3741	2,5066	16,2708
2D20-3	193,2593	19,8766	74,4048	57,1429	2,5078	16,3172
TRUNG BÌNH 2D20	186,3127	19,8514	71,7304	55,0889	2,5041	16,2496

GIAI ĐOẠN W=LIM =Lf/4Lc						
TÊN DẦM TN	TẢI TRỌNG P (KN)	ĐỘ VÕNG TN δ (mm)	M(TN) KN.m	Rku (ĐT) Mpa	Mở rộng vết nứt W (mm)	Biến dạng vùng kéo ε2 (‰)
2D12-1	70,9916	24,5266	27,3318	20,9908	3,1675	19,4724
2D12-2	61,4930	24,6681	23,6748	18,1823	3,1836	19,5065
2D12-3	68,9674	24,5982	26,5524	20,3923	3,1623	19,4277
TRUNG BÌNH 2D12	67,1507	24,5976	25,8530	19,8551	3,1711	19,4689
2D16-1	100,7653	24,5012	38,7946	29,7943	3,1608	19,6277
2D16-2	110,2422	24,6872	42,4432	32,5964	3,1689	19,7385
2D16-3	90,4048	24,5685	34,8058	26,7309	3,1670	19,5967
TRUNG BÌNH 2D16	100,4708	24,5856	38,6812	29,7072	3,1656	19,6543
2D20-1	176,1128	24,4216	67,8034	52,0730	3,1241	19,9028
2D20-2	185,7682	24,4255	71,5208	54,9279	3,1031	19,8397
2D20-3	190,0238	24,3683	73,1592	56,1862	3,1067	19,8894
TRUNG BÌNH 2D20	183,9683	24,4051	70,8278	54,3957	3,1113	19,8773

4.8. Analyse formula of flexural strength of the beam (σ_τ)

4.8.1. Comparison in bending resistance of tested beams and beams calculated by ACI-544 and Imam et al, Table 4.3

Table 4.3: Comparison in bending resistance

ĐỘ LỆCH GIỮA MÔ MEN THỰC NGHIỆM, MÔ MEN TÍNH THEO ACI-544 VÀ IMAM ET AL TẠI TRẠNG THÁI CỰC HẠN CỦA GIAI ĐOẠN KHAI THÁC W=0,3mm VÀ W=0,5mm							
tổ hợp dầm Thí nghiệm	Tại TTGH W(mm)	M(TN) KN.m	M(ACI) KN.m	M(IMAM) KN.m	M(TN)/M(ACI)	M(TN)/M(IMAM)	M(IMAM)/M(ACI)
D12	W=0,3	27,7145	18,6400	27,5971	1,4868	1,0043	1,4805
	W=0,5	30,1667	19,3728	28,8072	1,5572	1,0472	1,4870
D16	W=0,3	37,0079	24,9189	34,0955	1,4851	1,0854	1,3683
	W=0,5	41,7943	25,6965	35,3111	1,6265	1,1836	1,3742
D20	W=0,3	45,4508	32,9517	42,3188	1,3793	1,0740	1,2843
	W=0,5	61,0626	39,4825	49,3645	1,5466	1,2370	1,2503

** According to ACI-544 (ε_n=0,003) σ_t is calculated with a coefficient K=0,00772.

σ_t = 0.00772.(l_f /d_f)ρ_f F_{be} =0,00772 . (13/0,2) . 2 . 4,15=4,164 (MPa) and moment is calculated by the formula 4-1

** According to Imam et al 1995, fibre UHPC, grade ≤ 100MPa, is calculated with a coefficient K=0,0138 and:

σ_t = 0.0138.(l_f /d_f).ρ_f .F_{be} (MPa) = 0,0138.(13/0,2).2.4,15=7,444 (MPa), moment is calculated by the formula 4-1.

Therefore, in terms of bending resistant ability, experimental moment is higher than moment as specified in ACI-544 from 40% to 60%; and higher than the moment calculated by Imam from 10% to 23%. This proves that the experimental results are fundamental to modify formula to calculate σ_t .

4.8.2. Adjust coefficient K in formula 4-1 from experimental results

From formula 4-1:

Inferring:

$$\sigma_t = \frac{M_n - A_s.f_y.(d-\frac{a}{2})}{b.(h-e).(\frac{h}{2}+\frac{e}{2}-\frac{a}{2})} \qquad (4\text{-}2)$$

And from $\sigma_t = K \cdot (l_f/d_f) \cdot \rho_f \cdot F_{be}$ (MPa) (4-3)

Inferring: $K_{tn}=\sigma_t/\rho_f.F_{be}.(l_f/d_f)$ (4-4)

Results calculated in according to formulas from (4-1) to (4-4), the values M_{tn} σ_t, and coefficient K_{tn}, of the experimental beams at the specified points are presented in Table 4.4;

Table 4.4: Calculated results of coefficient K at the specified points

XUẤT HIỆN VẾT NỨT ĐẦU TIÊN W=0							
TÊN DẦM TN	TẢI TRỌNG P (KN)	ĐỘ VÕNG TN δ (mm)	M(TN) KN.m	σт (TN) Mpa	Biến dạng vùng kéo ε2 (‰)	Hệ số K(TN)	Độ lệch chuẩn S
2D12-1	39,2885	0,7701	15,1261	4,27	0,2582	0,00783	
2D12-2	36,0597	0,7910	13,8830	3,73	0,2369	0,00683	
2D12-3	37,8762	0,8810	14,5823	4,03	0,2489	0,00739	
TRUNG BÌNH 2D12	37,7415	0,8140	14,5305	4,01	0,2480	0,00735	
2D16-1	39,2988	0,7951	15,1300	2,43	0,2582	0,00444	
2D16-2	37,7876	0,9205	14,5482	2,17	0,2483	0,00398	
2D16-3	36,5826	0,8161	14,0843	1,97	0,2404	0,00360	
TRUNG BÌNH 2D16	37,8897	0,8439	14,5875	2,19	0,2490	0,00401	
2D20-1	47,8046	0,9905	18,4048	1,49	0,3141	0,00273	
2D20-2	51,4102	1,1526	19,7929	2,10	0,3378	0,00385	
2D20-3	56,7824	1,0682	21,8612	3,01	0,3731	0,00551	
TRUNG BÌNH 2D20	51,9991	1,0704	20,0196	2,20	0,3417	0,00403	1,79

Value of K* in average at the time of the first crack: K*=0,0051. This proves that at the time of first crack, steel fibre involves a very small load bearing, mainly depending on concrete and rebars.

GIAI ĐOẠN ĐỘ MỞ RỘNG VẾT NỨT W=0,3mm							
TÊN DẦM TN	TẢI TRỌNG P (KN)	ĐỘ VÕNG TN δ (mm)	M(TN) KN.m	σт (TN) Mpa	Biến dạng vùng kéo ε2 (‰)	Hệ số K(TN)	Độ lệch chuẩn S
2D12-1	76,2118	2,9840	29,3415	8,19	2,2719	0,01500	
2D12-2	67,8655	2,9781	26,1282	6,98	2,1957	0,01279	
2D12-3	71,8795	3,1300	27,6736	7,56	2,2716	0,01385	
TRUNG BÌNH 2D12	71,9856	3,0307	27,7145	7,58	2,2464	0,01388	
2D16-1	105,6623	3,0142	40,6800	9,95	2,4696	0,01823	
2D16-2	87,2755	3,2057	33,6011	7,35	2,4017	0,01347	
2D16-3	95,4351	3,0188	36,7425	8,51	2,3893	0,01558	
TRUNG BÌNH 2D16	96,1243	3,0796	37,0079	8,60	2,4202	0,01576	
2D20-1	99,3816	3,2848	38,2619	6,08	2,4885	0,01113	
2D20-2	115,7677	3,4535	44,5706	8,34	2,6015	0,01528	
2D20-3	139,0131	3,2646	53,5200	11,56	2,6706	0,02118	
TRUNG BÌNH 2D20	118,0541	3,3343	45,4508	8,66	2,5869	0,01586	2,30

Value of K* in average at W=0,3mm; K*=0,01516

GIAI ĐOẠN ĐỘ MỞ RỘNG VẾT NỨT W=0,5mm							
TÊN DẦM TN	TẢI TRỌNG P (KN)	ĐỘ VÕNG TN δ (mm)	M(TN) KN.m	σт (TN) Mpa	Biến dạng vùng kéo ε2 (‰)	Hệ số K(TN)	Độ lệch chuẩn S
2D12-1	84,4112	4,5603	32,4983	8,85	3,5869	0,01621	
2D12-2	73,2215	4,5210	28,1903	7,31	3,4653	0,01340	
2D12-3	77,4325	4,6782	29,8115	7,89	3,5467	0,01446	
TRUNG BÌNH 2D12	78,3551	4,5865	30,1667	8,02	3,5329	0,01469	
2D16-1	109,9689	4,6177	42,3380	10,00	3,7807	0,01831	
2D16-2	115,4441	4,7048	44,4460	10,74	3,7861	0,01966	
2D16-3	100,2568	4,6818	38,5989	8,69	3,7514	0,01591	
TRUNG BÌNH 2D16	108,5566	4,6681	41,7943	9,81	3,7728	0,01796	
2D20-1	133,1761	4,8216	51,2728	8,19	3,9400	0,01499	
2D20-2	155,8582	5,0537	60,0054	11,16	4,1451	0,02045	
2D20-3	186,7783	5,0871	71,9096	15,22	4,4425	0,02788	
TRUNG BÌNH 2D20	158,6042	4,9875	61,0626	11,52	4,1759	0,02111	2,57

Value of K* in average at W=0,5mm; K*=0,01792

4.9. Draw graphs (σ - ε); (σ-δ); (σ - w) from experimental results in accordance of SETRA/AFGC (as Figures from 4.5 to 4.8)

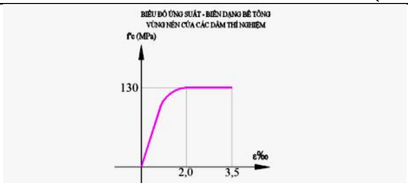


Fig 4.5: Graph of stress-strain at compression area of the tested beams

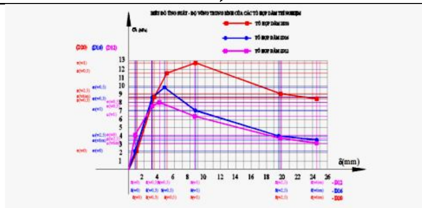


Fig 4.6: Graph of stress-deflection (σ-δ) of the tested beams

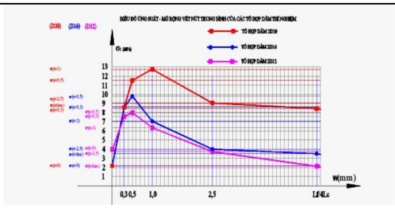


Fig 4.7: Graph of stress-opening width crack (σ- w) of the tested beams

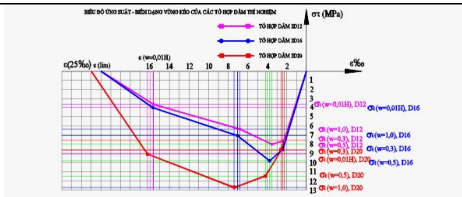


Fig 4.8: Graph of stress-strain (σ-ε) at tension area of the tested beams

Relation σ - ε is a fundamental used to calculate structures in according to SETRA/AFGC.

4.10. Apply to analyse bending behaviour of I33m beam

4.10.1. Methods to analyse bending behaviour of bridge UHSC beams in the world

Recently, in the world, there are three methods to calculate prestress beams casted steel fibre reinforced concrete. Method bases on guideline of SETRA/AFGC; method in accordance of DIN 1054-1; and method based on ACI-544.

It is possible to use rule of (p-w) in accordance of DIN-1054 (Germany), or use a relation $\sigma - \varepsilon$ of according to SETRA/AFGC (France); or use of block stress graph in accordance of ACI-544 of America.

The graph of stress-strain obtained from experimental results is used to establish to analyse bending behaviour of bridge beams and calculating agreed with ACI-544 with a maximum deformation of 10‰ as in Figure 4.9.

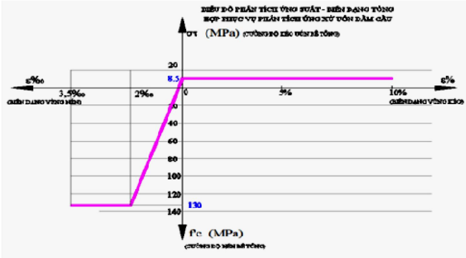


Fig 4.9: Graph of stress-strain from experimental results

4.10.2. Analyse of bending resistance of bridge beams using prestress UHSC grade 130MPa

+Formula

I cross section bended alongside, nominal bending resistant formula of the cross section can be determined as below:

$$M_n = A_{ps}.f_{ps}.\left(d_p - \frac{a}{2}\right) + A_s.f_y.\left(d_s - \frac{a}{2}\right) - A_s'.f_y'.\left(d_s' - \frac{a}{2}\right) + 0{,}8.f_c'.(b - b_w).0{,}65.h_f.\left(\frac{a}{2} - \frac{h_f}{2}\right) + \sigma_t.b_w.(h - e).\left(\frac{h}{2} + \frac{e}{2} - \frac{a}{2}\right) \quad (4\text{-}5)$$

+ Characteristics of the calculated beams, Table 4.5

Table 4.5: Characteristics of the calculated beams

Material's properties	**Unit**	**Notation**	**D33-40 (h=1650)**	**D33-70 (h=1650)**	**D33-130 (h=1650)**	**D33-130h (h=1100)**
Density of concrete	Kg/m^3	y_c	2500	2500	2500	2500
Compressive strength	MPa	f_c'	40	70	130	130
Flexural strength at the time of first crack in concrete	MPa	σ_τ	0	1,5	3,5	3,5
Flexural strength at the time of opening width crack of w=0,3mm	MPa	σ_τ	0	5,0	8,50	8,50
Maximum flexural strength	MPa	$\sigma_{\tau(max)}$	0	8,0	24,2	24,2
Elastic modulus	Mpa	E_b	30000	40000	50000	50000
Yield strength of steel rebars	MPa	f_y	350	350	350	350
Yield strength of steel fibre	MPa	$F_{sợi}$	0	2000	2000	2000

+Describe I cross section (includes I33m beam, h=1650mm in traditional and I33m beam with h=1100mm)

4.10.3. Calculation and results

* Check bending resistant ability in accordance of follow formula:

$$M_u \leq \phi M_n \quad (4.6)$$

* Check shear resistant ability in accordance of SETRA/AFGC as the follow formula:

$$V_n = V_{Rb} + V_a + V_f \quad (4\text{-}7)$$

*Condition $V_u < \Phi V_n$ (4-8)

* Check deflection of beam in accordance of TCVN 272-05 (calculate for beam D33-130h; h=1100mm), obtain following result:

Limited deflection [Δ]=L/800=40,375mm.

Assume that bridge structured from six beams with two lanes. Deflection distribution coefficient is 0.75. Then, deflection dues to moving load: Δ=16,97*0,75=12,75mm<[Δ], satisfied.

Calculated results for each beam as presented in Table 4.6.

Table 4.6: Calculated results for each beam

Parameter	D33-40 (h=1650)	D33-70F (h=1650)		D33-130 (h=1650)		D33-130h (h=1100)	
	272-05	272-05	ACI 544	AFGC	ACI 544	AFGC	ACI 544
α	0,85	0,8	0,8	0,85	0,85	0,85	0,85
Safety coefficient	1,43	1,3	1,3	1,25	1,25	1,25	1,25
f'_c	34	60	60	110,00	110,00	110,00	110,00
E	30000	40000	40000	50000	50000	50000	50000
σ_τ(w=0,3)	0	0	5	8,5	8,5	8,5	8,5
σ_τ(max)	0	0	8	24,2	24,2	24,2	24,2
β_1	0,75	0,65	0,65	0,65	0,65	0,65	0,65
b	2200	2200	2200	2200	2200	2200	2200
h	1650	1650	1650	1650	1650	1100	1100
bw	200	200	200	200	200	200	200
c	305,323	143,842	143,842	99,006	99,006	97,907	97,907
a	228,993	93,497	93,497	64,354	64,354	63,640	63,640
e	-	435,977	206,515	435,977	435,977	290,651	290,651
ΦM_n	1,19E+10	1,48E+10	1,41E+10	2,16E+10	2,08E+10	1,46E+10	1,52E+10
M_u	6,03E+09	6,03E+09	6,03E+09	6,03E+09	6,03E+09	5,52E+09	5,61E+09
$\Phi M_n/M_u$	1,96	2,46	2,33	3,57	3,44	2,64	2,70
Increased in comparison of		1,25	1,19	1,82	1,75	1,34	1,37

I33-40							
ΦV_n	1,57E+06	2,03E+06		2,67E+06		2,06E+06	
V_u	8,23E+05	8,86E+05	8,91E+05	9,88E+05		9,04E+05	
$\Phi V_n/V_u$	1,90	2,29		2,70		2,27	
Increased in comparison of I33-40		1,20		1,42		1,19	

From calculated results, draw graphs of ΦMn/Mu; ΦVn/Vu when changing grade of concrete and height of beam as presented in Figures 4.10, 4.11.

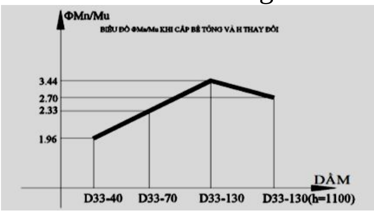
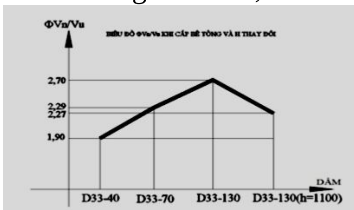


Fig 4.10: Graph of ΦMn/Mu when changing of grade of concrete and height of beam



Hình 4.11: Graph of ΦMn/Vu when changing of grade of concrete and height of beam

From investigated contents of Chapter 4, the following comments can be withdrawn:

- In experiment: Results obtained from 9 tested beams (dimensions of 125mm x 250mm x 2400mm according to ACI -544, drawn graphs of relations between load-deflection (P-δ); load-opening width crack (P-w); and stress-strain (σ-ε) to use for designing beam.

- Propose formula σ_τ setting up from experiments: $\sigma_\tau=K^*.(l_f/d_f).\rho_f.F_{be}$ (MPa), where K*=0,0159 -:-0,0179

- Building a calculated model used for bending behaviour of bridge beam as guided of Europe. Using model of ACI-544 and experimental flexural strength σ_τ from 8,5 to 9,65MPa when designing beam.

- Analyse bending behaviour I33 bridge beam used steel fibre, strength from 120 to 140 MPa, shown that it can be reduced the height of beam from 1.65 m down to 1.1 m (33% reduction) but maintaining bending, shear and deflection resistant abilities.

CONCLUSION AND SUBJECTION

1. CONCLUSION

From references and experimental investigation in UHSC, the author withdrawn the following conclusions:

The author collaborated with other members from University of Communications and Transportation (UCT) in the use of quartz rock from Thanh Son-Phu Tho and prepared sand and powder from the quartz rock. The quartz sand and powder agreed with international guidelines.

Using domestic materials to manufacture UHSC, grade from 120 to 140 MPa with proportion as below:

Table: Proportion of UHSC investigated

Cement	Quartz sand	Quartz powder	Silica fume	Superplasticiser	Steel fibre	Water
1	1,011	0,133	0,230	0,025	0,177	0,241

1.3. Experimental results showed characteristics of UHSC are as in the following Table:

characteristics	**Value**
Specified compressive strength (28 days) (MPa)	139
Specified flexural strength at the time of first crack (MPa)	12,06
Maximum specified flexural strength (MPa)	24,22
Elastic modulus (GPa)	$E_{đh}$=46,2 -:- 49,3
Slump (cm)	27
Flow (cm)	45- 64

1.4. Model of stress-strain used for calculation was built in accordance of Europe with specified compressive strength from 119-139 MPa, strain $\varepsilon 1$ = 2%, $\varepsilon 2$ = 3,5‰, elastic modulus: 46,2 – 49,3 GPa.

1.5. Experimental investigation in the work of reinforced UHSC beam, grade of UHSC 139 MPa, steel fibre R=2000 MPa, d=0.2mm, l=13 mm, fibre content 2% in volume leads to the following results:

Building graphs of relations of (P-δ); (σ-ε); and (σ - w) at points of nominal opening width cracks based on experimental results using for bridge design. Analyse bending behaviour, propose formula σ_τ setting up from experiments: $\sigma_\tau=K^*.(l_f/d_f).\rho_f.F_{be}$ (MPa), where K*=0,0159 -:-0,0179

Apply methods of calculation in bridge beams using UHSC as guided of (σ-ε) SETRA/AFGC and ACI 544 with $\sigma\tau$=8,5MPa.

1.6. Numbering analyse in bending resistance in accordance of limitation states of bridge beam structure with cross section of I, L=33m, using UHSC grade of 139 MPa, steel fibre content of 2%. This shows that the steel fibre improves bending resistance of beam 1.82 times, height of beam reduces from 1.65 m down to 1.1 m (33% reduction).

1.7. The above contents prove that it can be applied of UHSC in bridge structure. The experimental results can be used as references for researchers in UHSC.

2. SUBJECTION

It can be applied UHSC in bridge beam, casted bridge slabs or other partial members that need to specially strengthen in structure.

It can be used experimental methods, calculation models in design of bridge beam.

3. FURTHER INVESTIGATIONS

It is necessary to analyse structures using UHSC under impact and repeat loads.

In terms of structure, it is necessary to study behaviour of slabs and method to calculate slab structures on elastic foundation applying for special pavements.

It is necessary to investigate resistances of UHSC in radioactive, corrosion and erosion applying for special constructions.

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3. Prof. Dr. Pham Duy Huu; Dr. Nguyen Thanh Sang; Dr. Pham Duy Anh; Master Nguyen Loc Kha. Investigation in materials using for manufacture of UHSC. Journal of Transportation, No 07, July 2011;
4. Prof. Dr. Pham Duy Huu; Dr. Pham Duy Anh; Dr. Nguyen Thanh Sang; Master Nguyen Loc Kha, 2012. Report of project: Investigation in technology of manufacture of UHSC and application in bridge and high rise building structures (UHSFRPC);
5. Dr. Pham Duy Anh, Master Nguyen Loc Kha. Investigation in bending calculation of reinforced concrete bridge beam adding ultra strength steel fibre (UHPC), Journal of Transportation, No 03, 2013.