EXPERIMENTAL AND FINITE ELEMENT INVESTIGATION ON THE LOAD – SLIP BEHAVIOR OF COMPOSITE PUSH OUT SEGMENTS USING VARIOUS SHEAR CONNECTORS

by
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ABSTRACT

The study described herein deals with experimental and finite element modeling of a variety of composite steel-concrete column segments using shear connectors of different shapes and sizes to provide resistance to slip at steel – concrete interfaces . Hence , it represents a qualitative transition in the experimental and analytical investigations on shear connectors effectivity at steel – concrete interfaces , as most studies in the field of shear connectors were devoted to composite beam and slab – systems .

Three types of shear connectors – with four concrete grades for each type – were used in fabricating composite specimens . The twelve composite prototypes were subjected to push – out test individually to examine their behavior by measuring the slip values for each load incremental till failure , thus determining the resistance extent of each connectors type and specifying the failure mode at interface .

A nonlinear three – dimensional finite element analysis have been carried out on twelve composite column segments using **ANSYS** computer program (5 th version , 2002) to investigate their behavior and predict their load – slip relationships , equivalent stress distributions and concrete cracking patterns . The defined numerical modeling included using the eight node isoparametric brick element with smeared reinforcement (**SOLID** 65) and the eight node isoparametric steel brick element (**SOLID**45) , to model the reinforced concrete medium and the steel section and shear connectors , respectively , considering perfect bond between concrete and steel reinforcing bars . Nonlinear properties including cracking and crushing of concrete , yielding of steel section and reinforcement , and nonlinear bond – slip at interface were also considered .

Comparison of the experimental and theoretical results has shown good agreement that verifies the accuracy of the finite element model based on the smeared crack model of concrete .

Results have detected the development of the relative movement (slip) be at all ranges of the load-slip relationship at interface even with using effective shear connection and /or high quality concrete.

The headed studs have revealed the highest slip resistance and ultimate load over the channel and the L-shaped studs . The high strength concrete has also revealed the same superiority over the other three tested types of concrete .

التحري التجريبي وبالعناصر المحددة لسلوك الحمل- الانزلاق للقطع المركبة ذوات الأنواع المختلفة من روابط القص

الخلاصة :

تعنى الدراسة الموصوفة أدناه بالفحص المختبري والتمثيل بالعناصر المحددة لمجموعة متنوعة من قطع الأعمدة المركبة الحديدية – الخرسانية ذوات روابط القص بمختلف الأشكال والمقاسات لإعطاء مقاومة الانزلاق عند أسطح التلامس بين الحديد والخرسانة .

تمثل هذه الدراسة انتقاله نوعية في الأبحاث العملية والتحليلية في مجال فعالية روابط القص عند أسطح التملس الحديدية – الخرسانية لكون الأبحاث في مجال روابط القص منذ ثمانينات القرن الماضي ولهذا الحين – تختص في مجال البلاطات وأنظمة السقوف والعتبات المركبة .

تم استخدام ثلاثة أنواع من روابط القص – مع أربعة أنواع من الخرسانة لكل واحد منها - في تصنيع وتحميل وفحص قطع الأعمدة المركبة .

تم إخضاع نماذج الأعمدة المركبة الاثني عشر تلك إلى فحص الانضغاط push –out test بشكل منفرد لاختبار سلوك كل منها بواسطة قياس مقدار الانزلاق لكل قيمة حمل إضافية ولحين الفشل ، وبهذا يتم تحديد مدى مطاولة كل نوع من أنواع رباطات القص في مقاومة التحميل وتحديد شكل الفشل عند سطح التماس لكل نموذج .

تم استخدام تحليل لاخطي ثلاثي الأبعاد بطريقة العناصر المحددة باستخدام برنامج الحاسوب (ANSYS)(الإصدار الخامس ، 2002) في دراسة السلوك الإنشائي لقطع الأعمدة المركبة الاثني عشر واستقراء علاقة الحمل – الانزلاق لكل منها ، مع استقراء توزيع الإجهاد المكافئة وأنماط التشققات خلال الأوساط الخرسانية المسلحة ، يتضمن النموذج الرقمي المذكور استخدام العنصر الطابوقي الايزوبارامتري الخرساني ثماني العقد ذي التسليح الموزع (**SOLID** 65) ، العنصر الطابوقي الايزوبارامتري الحديدي ثماني العقد (**SOLID**45) لتمثيل الوسط الخرساني المسلح ، و مقطع الحديد الإنشائي ، وروابط القص على التوالي مع افتراض ترابط تام بين الخرسانة وحديد التسليح .

تم أيضا الأخذ بنظر الاعتبار تأثيرات الخصائص اللاخطية للمواد والمشتملة على التشقق والتهشم للخرسانة ، إجهاد الخضوع لمقاطع الحديد الإنشائي ولقضبان التسليح ، وظاهرة الانزلاق اللاخطي عند الترابط في أسطح التماس .

بينت المقارنة بين النتائج النظرية والنتائج العملية توافقاً جيداً وبذلك يتم إثبات دقة نموذج العناصر المحددة المستخدم والمبني على نمط التشقق المنتشر في الخرسانة .

أثبتت النتائج أيضاً وجود حركة نسبية (انزلاق) على امتداد أسطح التماس بين الحديد والخرسانة لغاية الفشل مع تحديد التطور عبر مراحلها لجميع النماذج الاثني عشر حتى مع استخدام روابط قص كفوءة او خرسانة عالية الخصائص .

أبدت روابط القص من نوع القضبان ذوات الرؤوس الكبيرة (headed studs) تفوقاً على النوعين الآخرين المستخدمين من روابط القص

(مقطع ساقية ومقطع L) في مقاومة الانزلاق والحمل الأقصى . كما أعطت النماذج ذات الخرسانة من نوع

خرسانة عالية المقاومة
(HSC) تفوقاً في نفس الخصائص – على الأنواع الثلاث الأُخرى من الخرسانة المستخدمة .

NOTATION:

A_{co} Cross sectional area of stud shear connector.

Ac Cross section area of concrete.

As Cross section area of steel .

Ast Cross section area of steel in tension .

E_c Concrete modulus of elasticity.

E_s Steel modulus of elasticity.

f_{cu} Cube compressive strength of concrete.

f_y Yield strength of steel beam .

G_s Shear modulus of steel.

h_c Thickness of concrete block

K_n Normal stiffness of connector.

Ks Stiffness of connectors per unit length (kN/mm2)

N Number of shear connectors

R.C Reinforced concrete .

S Spacing between shear connectors.

Ss Slip at the steel-concrete interface

β_c , β_o Shear transfer coefficient for closed and opened crack.

γ Slip.

ε Strain.

ε_y Yielding strain of steel plate.

ABBREVIATION:

LWAC Light weight aggregate concrete

NWC Normal weight concrete

FGC Fine grain concrete

HSC High strength concrete

CHBDC Canadian Highway Bridge Design Code

INTRODUCTION:

Numerous amount of research work was done in the filed of composite steel- concrete beams since the early fifties of the last century. However the most important ones in the last six years are denoted here.

In 2002, partial interaction analysis of beam - column members was made by Wu et al. **[1]** , by extending the classic linear elastic partial interaction theory to allow for axial forces and also boundary conditions associated with plastic hinges . Najem **[2]** , in 2003, carried out two theoretical models on partially layered beams to simulate a multi- layer beam with interlayer slip. The difference between the two models concerns the neglect (or regard) of both slip and separation between the layers of the beam . Validation of those models was verified by previous experimental evidentce. A model for predicting the stiffness behavior of the steel- concrete composite beam under negative bending at the serviceability limit state with different degrees of shear interaction was developed by Nile et al. **[3]** , in 2004 . Accuracy of the proposed analytical method in predicting deflection of a cantilever beam was verified on the bases of comparison with finite element modeling and experimental investigation. In 2005 Aziz **[4] ,** conducted a theoretical analysis and an experimental investigation of multilayer composite steel – concrete beams of partial interaction that permits slip and separation between layers. The developed method of analysis led to a set of differential equations for separation and slip which were solved numerically by the finite difference method **[4]** . Recently , in 2006 , the behavior of ten steel - concrete – steel sandwich (SCSS) beams of fully threaded bars (connected to upper and lower steel plates by nuts) as shear connectors was studied by Zebun **[5]** , in addition to twelve push-out tests using threaded bars of different sizes –as shear connectors . His experimental results showed that the procedure of calculating the ultimate load of sandwich beams based on plastic analysis can safely be used with a reduction factor of o.8 for the shear connectors ultimate force obtained from the push –out test **[5] .**

Parallel to the investigative effort defined above, several researches in the field of shear –connector behavior and push - out test have been performed . The most recent ones are highlighted herein.

Slobodan and Dragoljub **[6]** , in 2002 reviewed the most important analytic expressions for the strength of shear connectors with special attention to the recommendation given by the European standard Ec(4) **[7]**, and gave a commentary in that field. In 2004 , Deanna and Rambo –Roddenberry **[8]** , performed a comprehensive experimental study including 117 push- out tests on solid and composite slabs , whose results, along with 61 other beam tests were used to propose and veriefy a new stud strength reduction model. Experimental investigation on shear connection between steel and high strength light weight concrete (HSLWC) was performed by Valente and Cruz **[9]** , in 2005, to evaluate load-slip behavior . Larose et al **[10]** , conducted , in

2006, an experimental testing regime using push-out test specimens constructed with precast concrete panels connected to steel flanges with steel studs within a circular grout pocket to investigate the reduction in ultimate strength after cyclic loading. That study was done to examine the validity of embedding clustered shear studs in high strength grout for construction using precast deck panels according to the Canadian Highway Bridge Design Code (CHBDC) requirements . An investigation on the effect of confining reinforcement on the shear capacity and ductility of polypropylene concrete –steel composite systems was made by Maleki and Mahoutian **[11]**, in 2007 through several monotonic push -out test on this concrete type with channel shear connectors . Based on comparisons with result of the specified test setup comprising ordinary concrete specimens or polypropylene concrete specimens of confining reinforcement they concluded that the polypropylene fibers have no significant effect on the ductility of steel-plain concrete composite system , while stirrups in concrete blocks cause this property to increase . Lastly ,Thorsten **[11]** , studied the behavior of light weight aggregate concrete (LWAC) in composite specimens of headed studs in push-out test setup in 2008. He concluded that the empirical basic design rules given in EC(4) **[7]** , for headed studs in normal NWC underestimate the real shear resistance considerably in the case of LWAC. Hence it will be necessary to work out new design formulas for the application of LWAC **[11]**.

The privilege of the present study over the preceeding ones in the specified scope comes firstly from its embodiment to a wide and comprehensive experimental investigation of the shear –slip behavior for steel - concrete composite segments under push –out test including stiffness, ductility and ultimate resistance of those segments for three main types of shear connectors and for four types of concrete NWC , LWAC , FGC and HSC separately (thus forming twelve autonomous cases), and making comparisons to show effects of shear connectors and concrete types. Moreover, the present study has recorded an antecedence in the finite element modeling of such composite segments under push-out test using ANSYS finite element program which has given accurate results as in comparison with experimental results.

SHEAR TRASFER MEANS AND TESTING

<u>Definition</u>:

In steel-concrete composite structural members (primarily beams) the force applied to the interface between the two components is mainly, but not entirely, longitudinal shear. The interface is an origin of severe and complex stresses that require accurate analysis , therefore methods of connection have been developed empirically and verified by tests.

<u>Shear Connector Types and Behavior</u>*:*

Shapes of shear connectors in common use are specially established to provide adequate resistance to uplift as well as slip. They may be divided into two categories :(i) rigid connectors (bars or tees with hoops);and(ii)flexible connectors (studs and channels). The two groups differ in the mode of failure . While rigid connectors tend to cause higher stress concentrations in the surrounding concrete or even weld failure, the failure mode of flexible connectors is more consistent and less catastrophic as they derive their resistance essentially through bending of the connectors. Some commonly used types of connectors are shown in Fig 1. The rigid bar, tee and channel connectors are limited to shear transfer in one direction only ,while the headed stud

connectors can resist and transfer shear in any direction perpendicular to the shank, making it the more useful connector. Adding its simple fastening and little obstruction to reinforcement in concrete medium it has become the most common type in practice. It was recommended **[12]** , for that connector , that the shank diameter should not exceed twice the thickness of the steel part so that the full static strength of the stud can be utilized. In the present study the headed stud, channel and L-shaped types of shear connectors –shown in Fig1 **a**) and **c**) – have been used

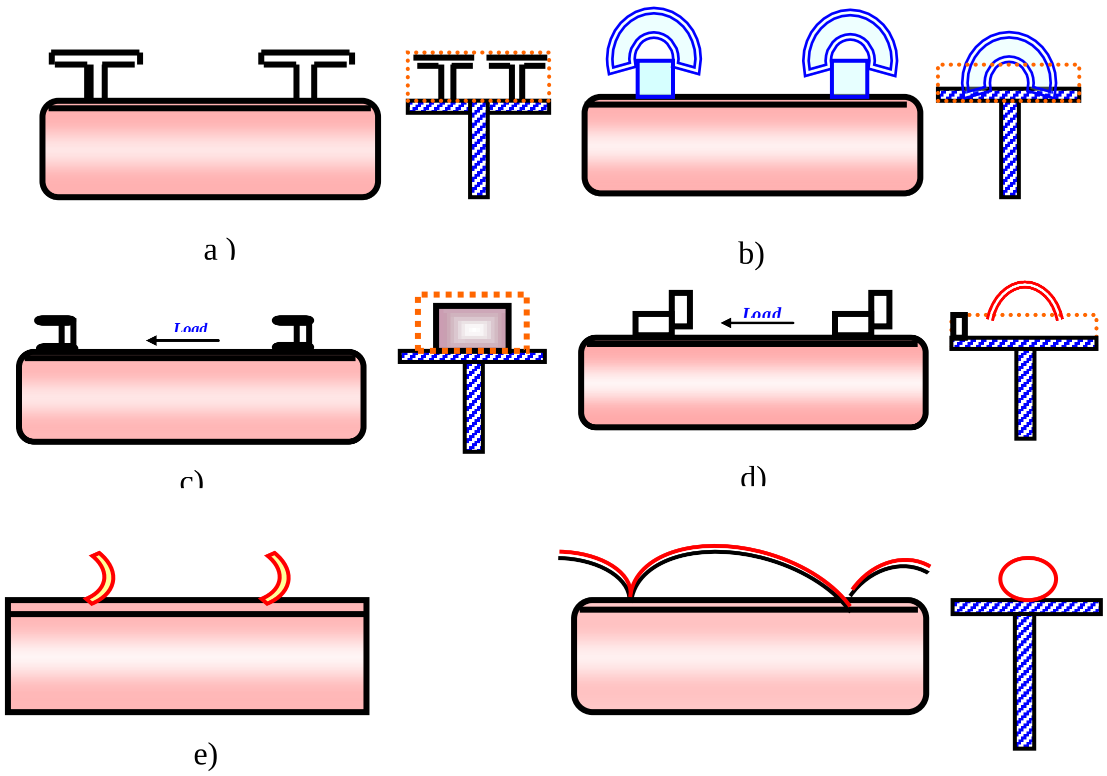


Fig.1: Types of Shear Connectors [4]

a) Headed Stud Connector , b) Bar Connector , c) Channel Connector , d) Tee Connector e) (L –shape Connector , f) Helical Connector .

Fig. 2 shows variation of the bearing stress on the shank of a headed stud connector with stress concentration near its base reaching four times the concrete cube strength owing to the concrete restraint there by the steel part, the shank of stud and the reinforcement [13]. The two major modes of failure are crushing of the concrete surrounding the connector (for sutds with large diameter)and connector shearing off at the base (for slender studs)

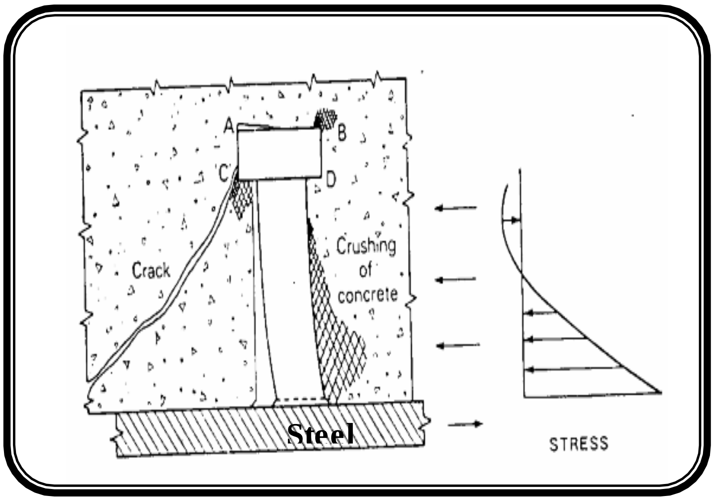


Fig.2 : Stress distribution on the Shank of a headed stud [13]

Push - out Test :

The shear connector property of most design relevance is the relationship between the transmitted shear force and the slip at interface. The load - slip curve should ideally be found from tests on composite beams . However , most of such data on shear connectors can be obtained from various types of " Push-out " tests shown in Fig. 3 in which the flanges of a short steel I-section segments is connected to two small reinforced concrete blocks [7, 12].

EXPERIMENTAL WORK :

Description and Classification of the Tested Prototypes :

With reference to Fig. 3 , the typical push –out test prototype of the present research work consists of a 560 mm long 254mm*147mm*43mm UB standard I-section connected at its two flanges to two 460mm *300mm*150mm reinforced concrete blocks by means of shear connectors welded to its flanges . The concrete blocks are reinforced by 10-mm diameter deformed steel reinforcing bars of mechanical properties shown in Table 1 . Their anchorage ends were made according to the ACI318-2002 sections 7.1 and 7.2

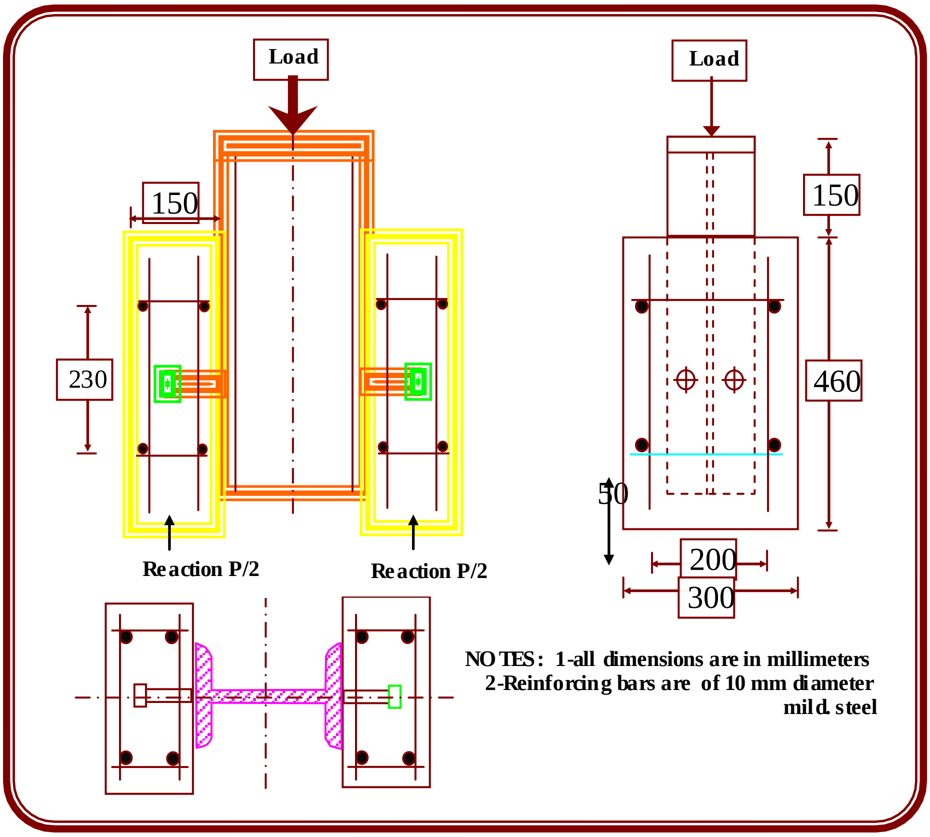


Fig .3: Schematic diagram for the typical push – out test prototype of the present work

i) Based on shear connector type :

headed stud shear connector containing prototypes : S- connector type .

channel shear connector containing prototypes : C - connector type .

L- shaped shear connector containing prototypes : L- connector type .

	Yield Stress (Fy) MPa	Ultimate Strength (Fu) Mpa	%Elongation at Rupture	Modulus of Elasticity (E) GPa
Ø10mm Reinforcing Bars	**285**	**510**	**20**	**203**
Ø19mm Smooth Bars for Stud L-connectors	**290**	**580**	**24**	**202**
UNP Channel for C-connectors	**240**	**540**	**24**	**205**
I-section	**248**	**400**	**17**	**205**

The S- and L- type connectors are made of 19mm diameter smooth bars whose lengths are 100mm and 210mm , respectively . Stud head of the S-type connector is of 30mm diameter . The C-type connector is a 60mm in length of UPN 100 standard channel section whose web depth , flange width and thickness are 80mm ,40mm and 4mm respectively . Mechanical properties of the specified steel bar and channel are given in Table 1 .

Any prototype of S-or L-type connectors contains two connectors –in one row welded to each flange , while each flange of any C- type connector prototype contains one connector only . Geometries , numbers , location and configurations of the three types of shear connectors are shown in Plate 1 .

Table 1 : Mechanical properties of the reinforcing bars , smooth bar and channel section used to form the three types of shear connectors , and the used steel I- section .

a) b) c)

Plate 1 : Typical prototypes before casting of concrete showing :
a) S- types connector , b) C- types connector , c) L- types connector

ii) Based on concrete type .
- Normal Weight Concrete (NWC) block containing prototypes .
- Light Weight Aggregate Concrete (LWAC) block containing prototypes .
- Fine Grained Concrete (FGC) block containing prototypes .
- High Strength Concrete (HSC) block containing prototypes .

Designations :

According to the two defined bases of classification , twelve different prototypes have been manufactured , fabricated , tested and analyzed in the present investigation .
Their designations are given herein :
SNWC, SLWAC , SFGC , SHSC , CNWC , CLWAC , CFGC , CHSC , LNWC , LLWAC , LFGC and LHSC .
The first character of each designation refers to the type of shear connector , while the remaining characters denote the concrete type .

Properties of Concrete Types and their Constituents :

General :

With reference to Table 2 the participated constituents of the used concrete types are the tap water , cement and natural silica sand .
Ordinary Portland cement (type I) [14] of chemical and physical properties conformable to the associated Iraqi specifications No.5 / 1984 [15] and presented in literature [16] was used thoughout this work .

Table 2 : Quantities of constituents and average compressive strength of cubes at 28 days for

		CONSTITUENTS AND QUANTITIES (Kg/ M^3)									
		Tap Water	O.P Cement	Ordinary Sand	Crushed Natural Gravel	Crushed Porcelinite	Silica Fume	Super Plasticizer	Silica Sand < 0.4mm	Average[1] Compressive Strength for Cubes at 28 days (MPa)	Modulus of Elasticity (GPa) [2]
CONCRETE TYPE	NWC	210	500	652	1022	0	0	0	0	44.2	31.26
	LWAC	210	500	500	0	520	15	2		39.55	29.56
	HSC	233	971.5	320	0	0	271.5	2	747	49	32.9

(1) Average cubic compressive strength for FGC at 28 days is 43 MPa .

(2) Concrete modulus of elasticity = 4700 (F_c) ^ $^{0.5}$, E for FGC = 30.82 GPa

The physical and chemical tests and the sieve analysis for the normal weight sand used as fine aggregate were all conducted in this work according to their associated specifications (17, 18 , 19, 20) Their results are shown in Table 3 .

Table 3 : Physical and chemical properties and grading parameters of the natural sand used in

	Property	Specification	Test Results	Award
1	Bulk specific gravity	ASTM C128-88[17]	2.55	Conformable to the associated specification
2	Absorption %	ASTM C128-88	2.1%	
3	Dry loose unit weight (Km/m3)	ASTM C29-89[18]	1600	
4	Sulphate content (SO_3)%	I.O.S No.45-84 [19]	0.14%	
5	Material finer than 0.075 mm%(*)	B.S882—1965 [20]	0.7%	
6	Fineness modulus (*)	B.S882—1965	4.3	

Normal Weight Concrete ***:***

Crushed natural gravel of 10mm maximum size was used as course aggregate for this type of concrete . Its specific gravity , absorption ratio and grading were all tested , measured and recorded [16] . They were found to be conformable to the I.O. S. No. 45-84 [19] .

From the present experimental work

*** These two properties are obtained from the sieve analysis done in this work [16]**

Light weight aggregate concrete :

Porcelinite crushed particles of 12.5 mm maximum size were used as coarse aggregate in this type of concrete to give a unit weight value of 1747 kg/m^3 . It also contains a superplasticizer and a hardener (as shown in Table 2) . Physical , chemical and mineral analysis for porcelinite were done by the State Company of Geological Survey and Mining " in Baghdad and were recorded [16] . They were conformable with the associated specifications (18, 21 , 22) . The grading of course porcelinite aggregate was conformable to ASTM(330-87) [23] as concluded from the sieve analysis results [16] .The " Tuf Flow 603 " superplasticizer used for (LWA , FG and HS)concretes in this work was classified into types (F) and (G) in ASTM – (494 (65) with properties presented in Ref. [16] .The hardener used in this mix is silica fume (SF) which is a pozzolanic material of very fine spherical particles and amorphous silicon dioxide . Its chemical and physical properties are documented [16] .

High strength concrete *:*

With reference to Table 2 , silica sand (SS) is the substitute constituent of porcelinite particles for the HSC mix relative to the LWAC mix . It is an inexpensive extremely hard fine granular material (0.15-0.40mm). Its chemical composition and physical properties are recorded [16] . Table 2 also refers to the rather low w/c ratio of HSC mix .

Fabrication of Prototypes :

The associated shear connectors for each of the twelve specified push –out tested prototypes previously described previously were welded to the two flanges at the appropriate locations by using the electrical resistance welding process of E70 electrodes [25] as shown in Plate 1.
The four concrete types were mixed in a horizontal rotary type mixer of 0.1 m3 capacity according to the ASTM C192-995[26] . Coarse and fine aggregates were first soaked and blended for sixty seconds in two – thirds of the required water quantity . For NWC , cement and rest of water quantity were then added and mixed for three minuted followed by additional three minutes of waiting for cement hydration . For LWAC , FGC and HSC Ohama's procedure [27] was followed by formerly mixing the required quantities of silica fume (SF) in dry state with the required quantities of cement for 15 minutes to ensure the thorough disperse of (SF) powder amongst the cement particles , then water was added with continuous mixing until uniform mix was obtained .
The four concrete mixes were then poured into their lightly oiled moulds which were vibrated for 30 seconds on a vibrating table . Except for NWC the risk of segregation is present by floating light weight aggregate particles which was extremely reduced by the application of silica fume . Curing of the prototype concrete blocks was realized by submerging in water . The 28 –day age average compressive strength values for the four concrete types were obtained from test results of three (150*150*150 mm) cubes for each of the NWC and LWAC mixes , and three (100*100*100mm) cubes for each of the
FGC and HSC mixes . Those Values are given in Table 2 .

Loading Scheme and Testing Procedure *:*

The twelve push –out tests were excuted using a testing machine of a loading frame and 2500 kN Capacity hydraulic jacks which was recently calibrated by the " Iraqi Central Organization of Standardization and Quality Control " . Each tested prototype was centrally loaded with two wood blocks placed under its two concrete blocks . 5-kN load was initially applied in order to settle down the loading system , then released within ten seconds . 3-kN incremantal loading series was then applied up to failure and the vertical slip value of the steel I – section relative to the two concrete blocks were measured at the level of shear connectors by means of calibrated dial gauge readings to 0.002 mm precision as shown in Plate 2 for each load increment .

The downward concentric load was applied to the end of the steel I-section by the crosshead of the machine acting through a ball seating , with care being taken each time in centering the load . The total duration of the test up to the failure point is about 45 minutes . If the prototype remained intact , loading was continued until severe cracking in the concrete block or fracture at the connectors occurred .
Measurements of the slip all the twelve push-out tests are plotted against the applied loads in figure to come later 7 and 8 . Values of the secant shear stiffness Ks for the twelve prototypes were calculated by dividing values of half's of the ultimate loads by the corresponding slip values and given in Table 4 that comes later .

Plate 2 : Typical prototype and push – out test machine with dial gauges attached to the concrete

FINITE ELEMENT ANALYSIS– "ANSYS" MODEL

Definition :

The commertial finite element package ANSYS [28] (Analysis System Version 5.4) was set up-with its parameters calibrated - and used in the analysis of the present twelve composite segments . The computer program has the capacity of solving linear and nonlinear reinforced concrete problems including the effects of cracking , crushing , shrinkage and creep of concrete , yield , placticity , creep , swelling , large deflection and large strain capabilities of steel sections , connectors and reinforcing bars , bond slip between shear connector (or steel rebar's) and the surrounding concrete medium , and temperature changes .

ANSYS Main Features

Concerning the present composite steel - concrete segments with steel shear connectors , the features for the program are specified :

Material Nonlinearities :

Nonlinearity properties of materials , namely cracking and crushing of concrete in tension and in compression , respectively and yield of the steel I-section , shear connectors and reinforcing bars were taken into account through ANSYS operations .

Element Types :

i) Steel I-section and shear connectors :

SOLID45 isoparametric element was used for three dimensional modeling of the mentioned steel parts . The element is defined by eight nodes each having three translational degrees of freedom as shown in Fig.4 .The element has plasticity , creep , swelling , large deflection and large strain capabilities .

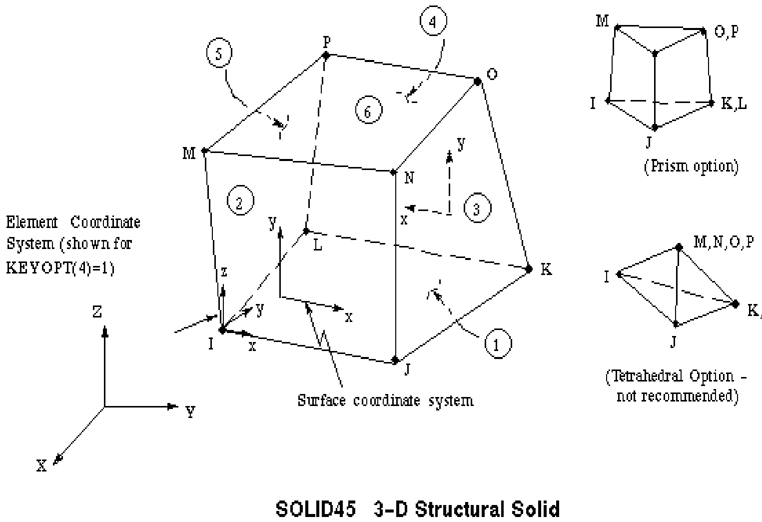


Fig. 4 : Solid 45 three dimensional structural solid elements [28]

ii) Reinforced concrete blocks :

SOLID65 isoparametric element was used for three dimensional modeling of the reinforced concrete blocks . The element is also defined by eight nodes each having three translational degrees of freedom as shown in Fig.5 . It is capable of plastic deformation , cracking in three orthogonal directions , and crushing . The steel reinforcement was introduced into this element by assuming it smeared throughout the element . Any orientation of the steel rebars is permitted . Use of this approach is supported by the fine – meshing of the concrete blocks , especially at locations of the reinforcing bars as recommended by the computer program [28] .

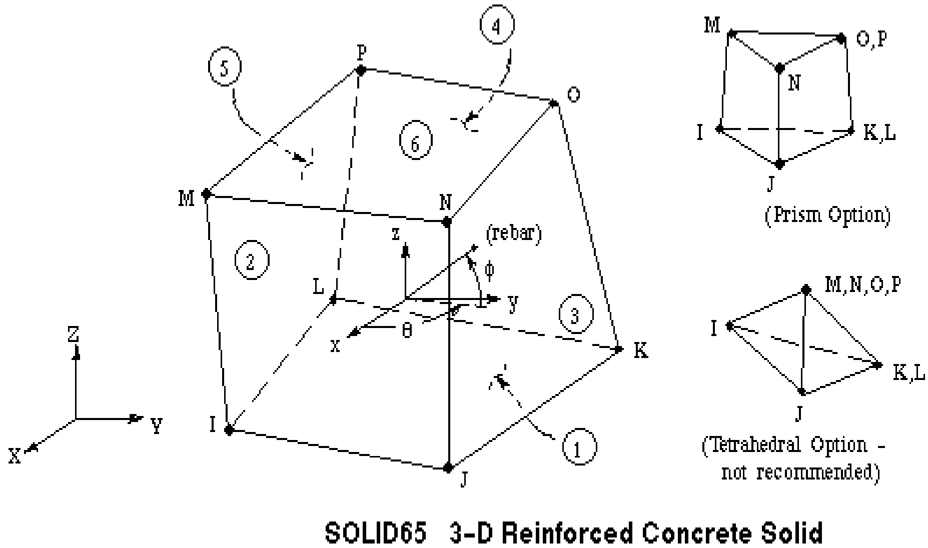


Fig. 5 : Solid 65 three dimensional structural solid elements [28].

Meshing :

Mesh controls of the computer program , if used , allow to establish the element shape , midside node placement , and element size to be used in meshing the solid model . This operation is one of the most important steps of the entire analysis . Volume elements (like SOLID45 and SOLID65) can often be either hexahedral (brick) or tetrahedral shaped , but a mixture of the shapes in the same model is not recommended . Hence , the tetrahedral shape was found to be suitable for meshing the steel I-section , shear connectors and the reinforced concrete block of the present composite segments .

Interfaces :

Bonds between the reinforcing bars (or the shear connectors) and the surrounding concrete of the two blocks of each prototype were all assumed to be perfect . Bond slips at those specified surfaces were then not allowed Accordingly , the defined contact locations were represented by participated nodes and the use of interface finite elements becomes unwarrantable .

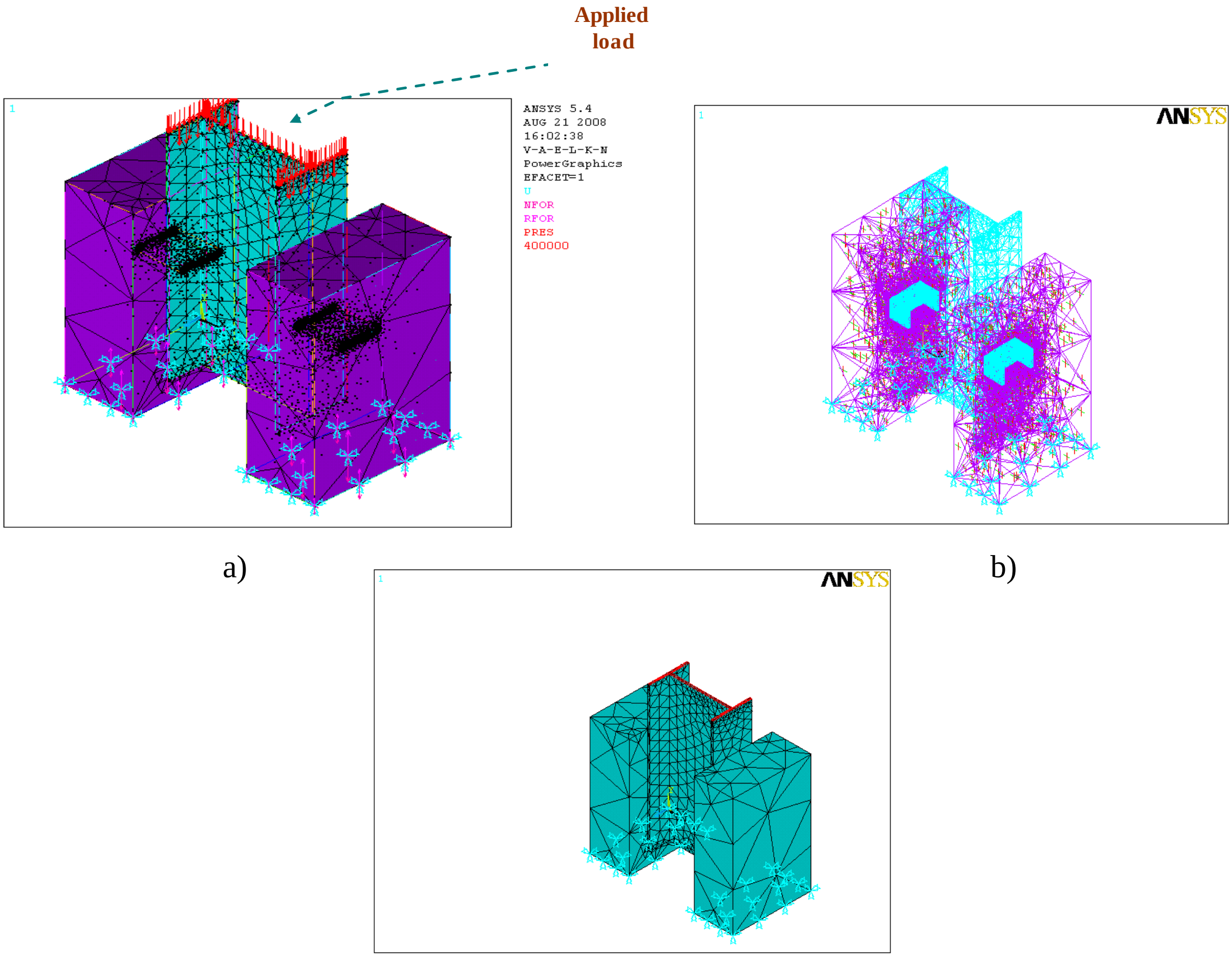
Nonlinear Solution Algorithm :

The nonlinear equations of equilibrium were solved using an incremental – iterative technique under load procedure . The full Newton –Raphson method was used for the nonlinear solution algorithm and the displacement criterion is used as a convergence criterion [28] .

Modeling of the Composite Prototype :

The steel I-section , the two concrete blocks and the steel shear connectors of each of the twelve studied composite prototypes were divided by ANSYS computer program into numbers of small tetrahedron SOLID45 and SOLID65 elements as appropriate . After load application , stresses and strains were calculated at integration points of those elements . An important step in the finite element modeling is the selection of the mesh density . The appropriate mesh densities for the twelve composite members were selected on the bases of a convergence study that determined the level of mesh reinforcement at which an increase in the mesh density was of a negligible effect on the results .

Based on that principle the degree of mesh refinement for each of the twelve composite segments was determined , then the number of the three dimensional elements for each component of the composite segment was obtained . The finite element mesh pattern, and the applied load and reaction simulations by ANSYS package for three typical composite prototypes comprising the three used types of shear connectors are shown in **Fig. 6 .**



a) b)

c)

Fig . 6 : Finite element meshes and the load and reaction simulations for three typical composite segments of the twelve prototypes comprising the three used types of shear connectors as modeled by ANSYS . a) case of headed stud . b) case of channel stud . c) case of L- shape stud .

PRESENTATION AND DISCUSSION OF RESULTS:

Outline *:*

The main objective of this study is to examine experimentally and assess theoretically the structural behavior and strength of concrete – steel – concrete composite segments with partial shear connection. Varying degrees of shear connection were realized by introducing different shapes and penetration depths of shear connectors and different types of concrete into the twelve composite segments subjected to push – out tests. The general behavior and test observations of those degrees of shear connection have then been discussed , with special attention to the correlation between the finite element prediction and the experimental evidence.

Inspection of Experimental Results :

General load – slip behavior :

Figs.7 and **8** explain the characteristics and stages of the load – slip behavior along test history of the twelve tested composite prototypes. Regardless of the difference between the degrees of shear

interaction (represented by the types of shear connectors and concrete) of the twelve prototypes, the test history (load – slip relation) can be subdivided into three ranges.
The first range was the serviceability stage in which the load increases almost linearly up to about 60 – 68 % of the ultimate (maximum) load and the vertical shear force acting at the steel – concrete interface was tramitted principally by the connector root . Because of the load concentration at the weld collar of the connector , only small deformations occured . Therefore , it is reasonable to regard full shear connection in that stage .
The second range extends from end of the previous range till the ultimate stage for which a significant nonlinear increase of the deformation was inspected . That loss of stiffness was caused by the local crushing of concrete around the foot of the shear connector and thus by a load distribution from the weld collar to the shank of the stud (or web of the channel –connector) , resulting in flexural and shear deformations of the connectors which depend wholly on the modulus of elasticity of concrete [11] . At this range the first cracks in the concrete blocks were observed .
The third range extends from the ultimate state till failure , where lower load levels were reached step by step . A ductile behavior was detected (especially for Stud and L-connectors) because as one stud failed , there was enough capacity in the neighbouring studs to absorb the load .

Effect of Shear Connector Type :

With reference to Fig. 7 , it is obvious that the stud – type connector revealed the highest values of the seviceability stiffness and the ultimate load capacity , followed by the L- type connector . Experimental work proved that the channel – type connector is the weakest in the two defined flexural parameters , which may primarily attributed to its relatively narrow cross-sectional dimensions and penetration depth , then to its be inability to absorb the load shed after the first yield (as only one channel – type connector was attached to a flange) . Percentages of excess in the values of the ultimate load capacity for the tested composite prototypes with Stud- type connectors over the corresponding values for the associated prototypes with L- type connectors were 40% , 20% , 29.5% and 57% , for NWC, LWAC , FGC and HSC concrete types , respectively , while the corresponding percentages of excess for the case of Stud – type connectors relative to the channel – type connectors were 78% , 40% , 65% and 92% for NWC , LWAGC , FGC and HSC concrete types , respectively .

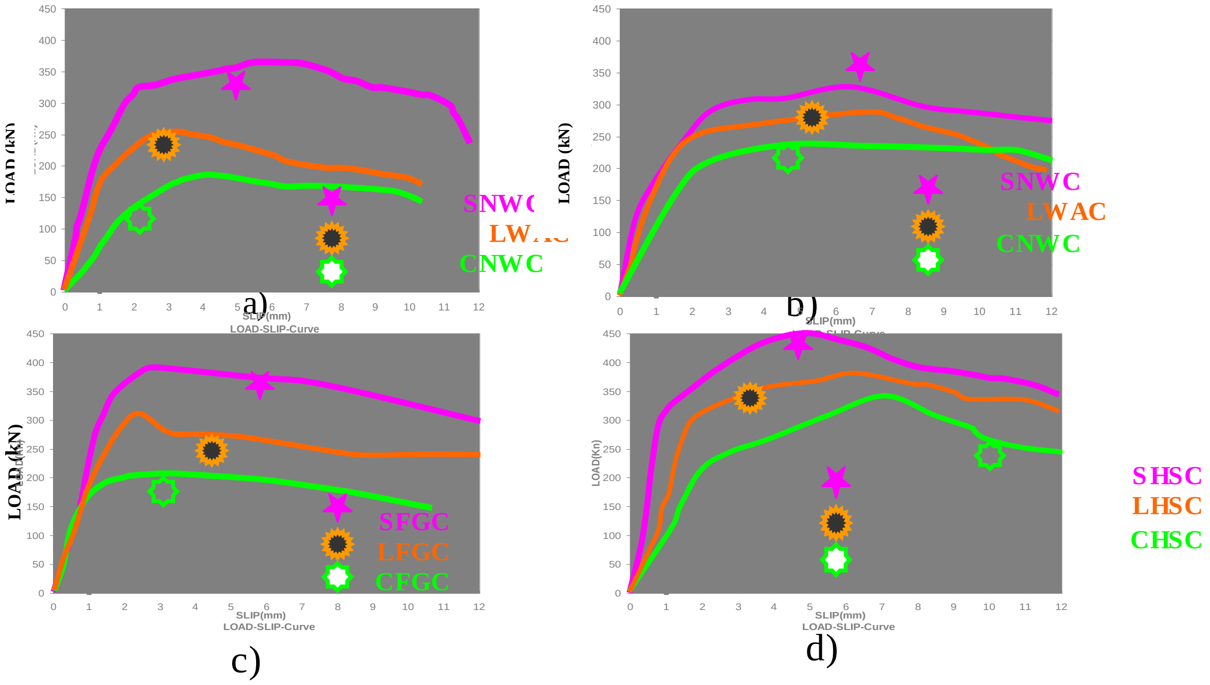


Fig.7 : Load – slip relationships obtained experimentally for the three tested types of shear connectors embedded in each of the four used types of concrete . a) in NWC, b) in LWAGC , c)in

EFFECT OF CONCRETE TYPE:

Recalling the difference between values of the E- modulus for the four used concrete types NWC , LWAGC , FGC and HSC, the inspection of **Fig.8** (which shows the load – slip relationships for each type of shear connectors embedded in the four concrete types in one graph for the sake of comparison) leads to the following remarks :

i) In general the same trend of this relationship (formerly subdivided into three ranges) is detected for the four concrete types .

ii)In the first (serviceability) range of that relationship the behavior of the two stronger types of concrete (NWC , and HSC) under working loads differ only irrelevantly from those of the two less strong concrete types (LWAGC and FGC) because of the bigger initial – stiffness of the two stronger types by about 40-56% . This fact is not of great significance because of the small deformations at this load level .

iii) In the second range (extending from first yield up to ultimate stage) wide difference is shown in the development of the load - slip curves for the two weaker concrete types (LWAGC and FGC) relative to the two stronger types (NWC and HSC) . This is mainly because of the loss of stiffness due to local crushing the concrete around the foot of the shear connector .

iv) In the third range (from the maximum load stage till failure where lower load levels were reached gradually) similar wide difference between development of the relationships for the two weaker and the two stronger concrete types resulted . This is mainly due to the difference in the elastic bedding between the cases of the stronger and the weaker concrete types [11] .

From another point of view the effect of concrete type on load-slip relationship can be evaluated on the bases of the **shear stiffness parameter Ks** value which is defined as the load –slip ratio at a load level of 50% of the ultimate load , thus representing slope of the secant stiffness at that load level of the relationship and giving an indication to the average load- slip relationship and the level of the ultimate load . Referring to **Table 4** (which gives values of the Ks parameter for the twelve tested prototypes) it can be noticed that the effect of concrete compressive strength (which decreases monotonicly for HSC to NWC to FGC to LWAGC by the evidence of Table 2) on the Ks parameter is vital . In its absolutely maximum particular case the reduction in the Ks parameter reaches 66% when the concrete type is changed from HSC to LWAGC with channel – type connector .

Table 4: Variation in the values of shear stiffness parameter Ks for the twelve tested prototype) relative to the NW concrete type for each of the three types of shear connectors obtained experimentally

Connector type used with concrete type	50%Ultimate load (kN)	Slip corresponding at 50% (mm)	Ks (kN/mm)	% difference (Ks) with in NWC
SNWC	196.8	0.734	268.122	0
SLWAC	165.2	1.48	111.623	-58.4
SFGC	174.7	1.21	144.38	-46.2
SHSC	219.3	0.64	342.65	27.8
LNWC	152.1	1.01	149.7	0
LLWAC	138.8	1.76	78.86	-47.3
LFGC	98.4	1.31	75.12	-49.9
LHSC	174.4	0.82	212.7	42.08
CNWC	112.5	1.18	95.334	0
CLWAC	89.6	1.88	47,658	-50
CFGC	103.3	1.42	72.7	-23.75
CHSC	144.6	1.04	139.03	45.8

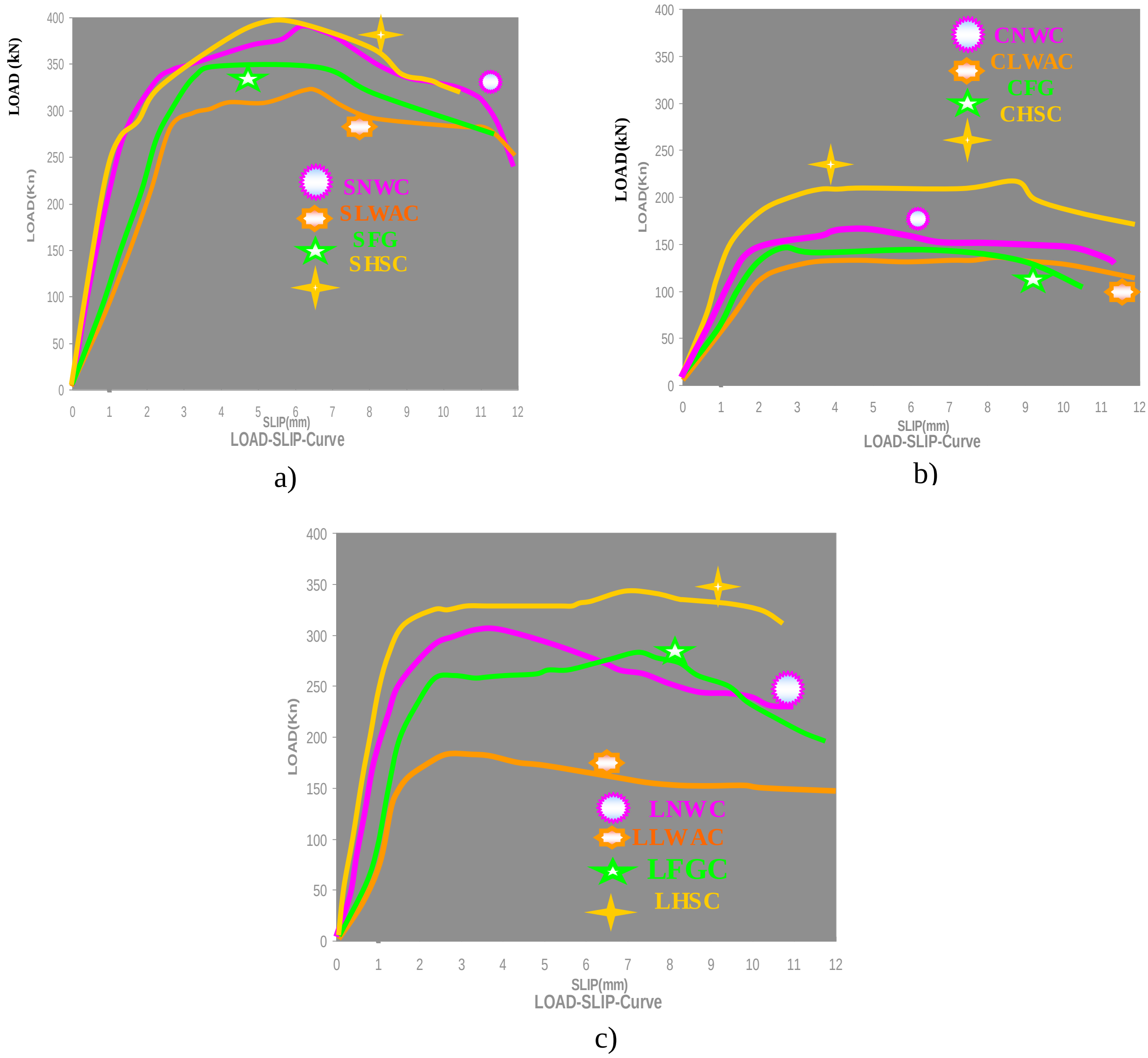


Fig. 8 : **Load – slip relationships obtained experimentally for the four used types of concrete with each of the three tested types of shear connectors :**
a) with Stud-type connector , b)) with channel-type connector , c) with L-type connector

Correlation between Finite – Element Prediction and Experimental Evideuce :

With reference to Fig. 9 which shows the load – slip relationship determined by the ANSYS model (according to the formerly mentioned concepts and fundamentals) beside the corresponding results obtained from the present experimental testing for each of the twelve composite prototypes , the following concluding remarks are recorded :

i) Good agreement between the experimental and numerical investigations in describing and evaluating the charactaristics of the first stage (linear serviceability range) is obtained , mainly the initial stiffness .

ii) The numerical model was accurate in predicting the ductile elasto-plastic trend of the load – slip relationship . This remark was confirmed by the complete coincidence between the experimental and theoretical values of the deflection at failure for all prototypes .

iii) The numerical model gave no distinct transition from the second to the third fiormerly defined ranges of the studied relationship .

iv) According to the last remark no distinct ultimate stage point was detected from the load – slip relationship of the finite element model , leading to a noticable difference in the maximum load resistance values given by the experimental and theoretical analysis .

v) The unability of the present numerical model in describing the third range of the load- slip relationship (beyond the maximum load stage till failure load where low load levels were attained gradually) , has led to failure values not in coincidence with the corresponding experimental ones .

vi) The view drawn from Tabel 5 , which shows that the percentage difference between the theoretical and experimental values of the Ks factor , is fairly positive . The maximum defined difference is 19.05% for SHSC prototype (the absolutely strongest one) , while the minimum difference is 3.318% for CLWAC prototype (the absolutely weakest one) . The avarege difference for the twelve prototypes is 7.58% , a quantity reflecting the accuracy of the present finite element model in simulating the tested composite segments .

Table 5 : Experimental and theoretical values of the shear stiffness parameter Ks and the percentage difference between them for the twelve studied composite segments .

		Prototype Designation											
		SNWC	**SLWAC**	**SFGC**	**SHSC**	**LNWC**	**LLWAC**	**LFGC**	**LHSC**	**CNWC**	**CLWA**	**CFGC**	**CHSC**
Shear stiffness parameter kN/mm	**Experimental**	268.12	111.62	144.38	342.65	149.7	78.86	75.12	212.7	95.334	47.658	72.7	139.03
	Theoretical	252.9	107.2	138.8	323.6	142.87	74.65	70.001	202.09	89.7	44.34	69.123	131.8
	%difference	15.22	4.2	5.58	19.05	6.83	4.03	5.119	10.61	5.63	3.318	3.577	7.23

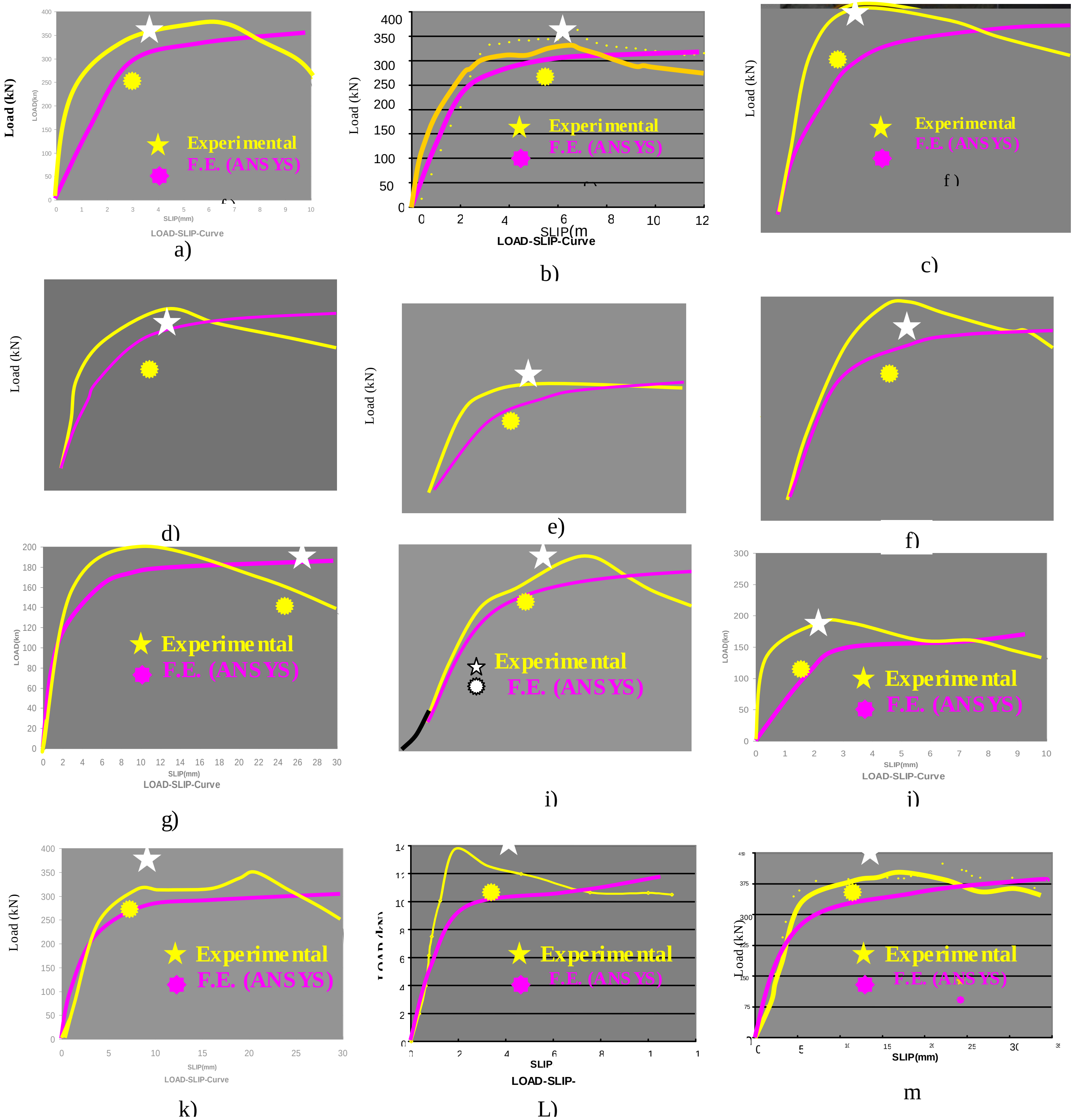


Fig. 9: Experimentally and theoretically obtained load -slip relationships for the twelve studied composite prototypes : a)SNWC , b) SLWAC , c)SFGC d) SHSC , e) CNWC , f) CLWAC , g) CFGC , h) CHSC , i) LNWC , j) LLWAC , k) LFGC , l) LHSC

CONCLUSIONS :

The following conclusions have been reached in this study so far :

- Regardless of the degree and state of partial interaction at the steel-concrete interfaces the twelve tested composite prototypes have given the same general trend , features and characteristics of the load – slip relationship consisting of the main three ranges detected in few recent experimental researches , especially the linearly , high stiffness and precise limit of the serviceability stage (60-68%) of the ultimate load).

- Further comparative inspection of the load-slip curves for the twelve tested composite prototypes shows the significant and quantitively - organized differences between their ultimate load values at ends of the second ranges of their curves accompanied by slight differences in the corresponding slip values .

- The comparative inspection also shows the same general signpost of differences in levels of load – slip curves in the third range (from ultimate stage till failure) for the twelve tested prototypes . More significant differences in the extensions of slip beyond ultimate stages for the twelve prototypes are attained , referring to significant differences in ductility .

- The shear connector shape , penaration depth and configuration are of major effects on serviceability stiffness and extension , ultimate load capacity , and ductility of the shear in partially interacted steel- concrete composite segments . The S- type connector comes in the vantage followed by L-type one .

- Quantitatively speaking with regard to connector type effect on ultimate load capacity of composite segments , the average excess percentages in that property values for segments with S-type connectors over segments with L- type ones are 48.5% and 24.8% for the stronger (NW and HS) and for the weaker (LWA and FG) types of concrete , respectively . The corresponding average excess percentages for the case of S-type connectors relative to C-type ones are 85% and 52.5 % for the stronger and weaker types of concrete , respectively .

- The first (serviceability) range of load- slip relationships for the composite segments revealed relatively slight effect of the concrete elasticity modulus (E-value)on stiffness of segments under working loads (in spite of the rather large differences in the initial stiffness values of about 40-56% between the two stronger and the two weaker tests types of concrete . This is due to the small deformations at the service load levels .

- The ultimate compressive strength of concrete directly affects the development of the load-slip curve in the second range (from end of service stage to ultimate stage). The higher the difference in concrete strength this property values of concrete types , the wider are the differences between levels and extensions of those curves in that range , a criterion confirming the prediction of stiffness loss due to local crushing of the concrete surrounding the connector foot .

- Similar wide differences between levels and extensions of the load –slip curves for the two stronger and the two weaker tested types of concrete are obtained .

- The shear stiffness parameter Ks of a composite segment of partial shear interaction at steel-concrete interfaces is solely affected by the concrete compressive strength . The present work shows that 66 % reduction in Ks value is attained when replacing HSC by LAWC in composite segments of C- type connectors .

-In regard to the correlation between the finite element prediction and the experimental issue for the steel – concrete segments of various types and levels of partial shear interaction at interfaces , the numerical model has proved its efficiency and accuracy in predicting characteristics of the serviceability stage and the ductile elasto- plastic trend of the load-slip relationship , which appears precisely in the highly accurate computation of slip value at failure .

- On the other hand , ability of the present finite element model in giving distinct border between the second and third range of the load-slip relationship and in protruding the ultimate stage was limited , this has led to discrimination between the third range of the load-slip relationships of the numerical model and the experimental work .

– The present finite element model has proved its efficiency in computing the shear stiffness parameter Ks values as compared with the corresponding experimentally obtained values . 7.58% comprehensive average difference between the two methods is attained .

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