INTERFACE DEPENDENCY OF REINFORCED CONCRETE JACKETING FOR COLUMN STRENGTHENING

K. Sakil Ahmed[1] and Riaz Mahmud [2]
[1,2] Civil Engineering Department,
Military Institute of Science & Technology,
Mirpur Cantonment, Dhaka 1216, Bangladesh.

[1]< drksa@ce.mist.ac.bd >, [2]< riazeity7239@gmail.com>

ABSTRACT
This paper investigates the structural capacity enhancement of reinforced concrete column by concrete jacketing. In view of investigating structural capacity and integrity of jacketed column, a total twelve retrofitted column samples with different jacket thickness were experimentally tested. Samples are prepared with different types of interface including no surface treatment, addition of bonding chemical, roughening of old surface, application of welded ties, and changes in clear cover to investigate the influence of interface bonding between new and old concrete. Analytical equations for jacketed columns are formulated as per elastic principle maintaining strain compatibility at the interface in addition to concrete modeling using finite element method (FEM). Column interaction diagrams as formed by analytical equations are compared and verified with existing Japanese code and available FE software packages. Experimental investigation shows that failures occur relatively earlier at the column interface than the core of the retrofitted column. Comparative study in terms of interaction diagram shows that experimental result well agrees with the computed analytical result but deviates from the FE analysis. Finally, an interface bonding reduction coefficient in the range of 0.65-0.88 is proposed for RC jacketed column subjected to different types of interface.

Key Words: Column Jacketing; Interface bonding; Analytical equation, Interaction diagram; Finite element analysis.

1.0 INTRODUCTION
The demand of strengthening for reinforced concrete structure is rapidly increasing since early 2000 in Bangladesh. Previously, many buildings of the country were designed neither following any standard guideline nor considering lateral loads.

In order to avoid potential earthquake hazard, latest Bangladesh National Building Code (BNBC)-2014 guideline also demands more structural resistance that suggests to strengthen many existing building structures of Bangladesh. Recent earthquakes, Rana Plaza incident and some other structural hazards raised the attentions towards the structural strengthening. Apart from seismic demand and poor design, changes in live loads and user facilities, deterioration of the load carrying elements, design errors, poor construction quality during erection, and aging of structure force the users to strengthen the structural elements. However, there is no clear guideline in existing local code to investigate the column retrofitting.

Mander et al. (1988) introduced stress strain model for confined concrete to determine the confined capacity and influence of confinement (Mander et al. 1988). Later, Park and Rodriguez (1994) conducted a number of experiments on column jacketing to investigate the confinement effect on column capacity (Rodriguez and Park). Their study revealed that jacketing and confinement significantly improve the column capacity. Some researcher has already contributed in the development of steel jacketing (Belal et al.

2015; Chen and Teng 2001). Their research work concludes that the steel jacketing may be used as one of the way to strengthen existing structural element. However, the interface between steel and existing concrete is to be taken critically since the behavior is not always as composite. Concrete jacketing by fiber reinforce polymer or carbon fiber have been conducted extensively over the years by many researcher (Brownjohn et al. 2001; Buyukozturk et al. 2004; Hassan and Rizkalla 2003; Karayannis and Sirkelis 2008; Malek 1998; Sheikh et al. 2002; Smith and Teng 2002; Wu et al. 2006) . Some research work are also found in this area dealing with seismic loading (Wu et al. 2006; Xiao et al. 2012; Xiao and Ma 1997).

The existing literature demonstrates that more research work have been conducted on retrofitting using steel jacket or FRP. Though there are some studies on RC column jacketing, there is no research on improving surface treatment and its contribution in capacity enhancement. Still there is clear gap in RC column jacketing which is commonly used in third world countries where labor cost is comparatively lower than the material cost. It is worthwhile to mention that the country is rapidly developing new structures as well as strengthen in existing buildings and there is no clear guideline on this technique due to lack of research attention in this area. In addition, there is no authentic analytical research in existing literature to determine capacity enhancement through RC jacketing.

In this circumstances, this paper investigates the structural capacity enhancement of column by RC column Jacketing. In view of detail investigation, both experimental and analytical studies were conducted to determine actual load carrying capacity of jacketed column under axial loading. Based on the study, analytical equations are proposed to estimate jacketed column capacity. Results in terms of column interaction Diagram formed by derived analytical equations were also compared with that of derived from Japanese retrofitting codes and FE model in ETABS 2015 & SAP 2000 v17.

2.0 EXPERIMENTAL INVESTIGATION

2.1 Detailing and Preparation of Sample

In this study, a total twelve column samples were prepared with different dimensions and various jacket thickness. Due to the height constraint of the Universal Testing Machine (UTM) the samples are 3 times smaller than the actual column size. Generally, the height of the short column samples were kept to 800 mm with a cross section of 102 mm x 102 mm for Reference Sample (RS) that represents the existing column. Local experience on the building of Dhaka city provides the information that the compressive strength of existing concrete of old building is very low which is mostly in the range of 1500-2000 psi. In view of modeling weak concrete, this study also employs weak concrete for reference section where comparatively higher strength is used for jacketed concrete as widely practice in the country. Clear covers were changed to 12 mm and 15 mm for 25 mm and 31.5 mm jacket thickness, respectively. The other key features of the samples are presented in Table 1.

Table1: Experimental Samples and properties Sample Features

Sample Features	Properties /Dimensions
1. Sample height	800 mm
2. Cross Section (Actual Column)	102mm×102 mm
3. Main Bar diameter	8mm
4. No of Main bars (Actual Column)	4
5. No of Bars after strengthening	8 (in addition to 4)
6. Tie Bars Diameter	2.5mm
7. Tie Spacing	100mm
8. Clear cover	12 mm to 15mm
9. Jacket thickness	25 mm and 31.5 mm
10. Stone Chips Size	12.7mm Down-grade
11. Fineness Modulus	1.2 -2.2
12. Steel Grade	40 (f_y=40,000 psi)
13. Chemical for Surface Treatment	Epoxy, Master Flow
14. Application of Load (UTM)	Displacement Control
15. Mixing Ratio Sample	1:2:4 for Existing 1:1.5:3 for Jacketed sample

This procedures and sample properties are well agreed with the previous studies conducted by (Buyukozturk et al. 2004; Chen and Teng 2001; Karayannis and Sirkelis 2008; Rodriguez and Park)

2.2 Surface Treatment

In order to maintain proper mechanical interlocking between old and new concrete, surface treatment were conducted in most of the samples. In addition to achieve proper monolithic action between old and new concrete, surface of reference column was roughened by sand blasting with hand chisel all over the surface as shown in Fig.1. Two among twelve samples were also treated with chemical bonding agent. For chemical bonding, in this study master flow was used supplied by BASF. This chemical originated by Germany is widely used in many countries of the world. In addition, tie bars are applied to connect between old and new concrete for some samples.

2.3 Test setup

The machine is well equipped with computer monitor, automatic dial gauge and also adjust to extensometer which is capable to conduct test in both load control and displacement control procedure. In this study, we consider displacement control mechanism in which load was applied at a rate of 1 mm/ min rate. In the test setup, additional steel base plate of 225 mm x 225 mm cross section and 25 mm thick is fixed at the two ends so that forces can be uniformly distributed on the specimen and hence to examine the performance of the column under pure compression. Samples were placed in the base along with base plate at both ends which represents pin ended supports as shown in Fig. 2.

Fig 1. a. Reference Sample. b. Surface roughening c. Sand blasting. d. Application of epoxy bonding. e. Jacketed framework. f. Application of jacket before casting

Fig. 2: a. UTM b. Sample placement c. Load application and monitoring by dial gauge and software

2.4 Failure Patterns of Tested Samples

In the test all samples were subjected to uni-axial compression. Samples were designated with a prefix according to their jacket thickness. It is observed that with the application axial load on the samples longitudinal cracks were formed and they collapse as those cracks progress to larger. The failure patter of the most of the samples confirms that usually samples fails with generation of longitudinal crack to the full and partial depth as shown in Fig.3. In order to understand the interface type inside the column samples, suffix 'N', 'B', 'M', 'W' and 'C' were used. The representation of those suffixes are 'N'-no bonding agent, 'B'-surface prepared and bonding agent used, 'M'- monolithic casting, 'W'-welded ties, and 'C' change of clear cover.

Few lateral cracks were also observed in the failed column sections. Local failure occurred due to stripping out of concrete at the new and old concrete interface. This happens as the jacketed part of the column is unable to maintain strain compatibility at the interface between new and old concrete. Crushing of jacket concrete along with failure of jacketed ties were seen. Buckling of rebar occurred in the sample where ties were not welded. In some samples, new concrete failed at the corner due to high stress concentration which also agrees with existing literature.

Fig 3. Typical failure pattern of sample with their designation below

2.5 Structural Behavior and Interface Influence

All column samples are tested under uniaxial compression. Since the sample sizes may not be exactly same, stress vs strain relationships are presented as structural output

as shown in Fig.4. The result shows that the stress-strain curves for different samples varies according to their interface type (such as surface treatment, monolithic casting, bonding agent or welded tie) and the pick stresses were found in the range of 10 MPa to 17 MPa approximately. In addition, the ultimate strain of confined concrete increases due to jacketing as tensile reinforcement undergoes strain hardening which is well agreed with (Mirmiran et al. 1998; Saatcioglu and Razvi 1992).

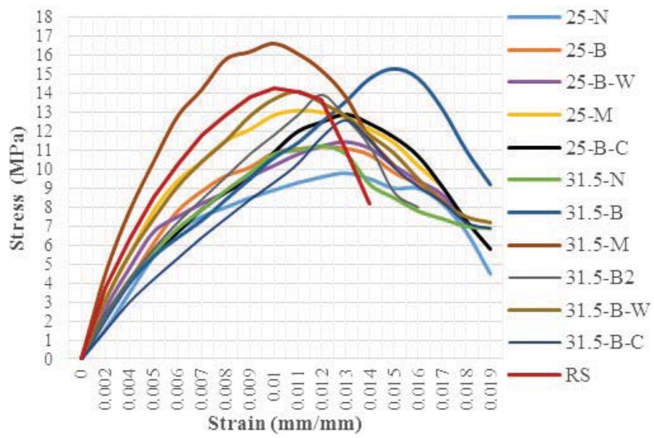

Fig 4. Stress vs. Strain for different Column samples

Effect of surface preparation and bonding agent

The result shows that the sample containing no surface treatment or bonding agent at the interface 25-N failed in the minimum stress (10MPa). However, if surface treatment is confirmed in the sample with chemical bonding, the higher pick stress can be achieved which can as high 85% of that of the monolithic casting (Sample 25-M and 31.5-M). It is important to note that monolithic casting will give strength of short column as size increase where the increase percentage is 100%. The influence of bonding is presented in terms of coefficient of bonding CB. The result shows that if no chemical agent or surface treatment is used the value of CB is in the range of 0.25-0.30 with respect to monolithic casting. It affected the failure pattern of sample as described above. It is also clearly observed from the curves that if clear cover is increased and welding is confirmed between old and new bars, the pick shift in larger strain that also ensure more ductility for the column. This influence of "clear cover change" may be adjusted with more tie bars in the retrofitted column.

As per failure pattern, welding of ties diverted the failure criteria from buckling of main and tie bar. It was found from Fig.4 that welded tie increases the strain rate than that of non-welded ties in addition to increase of axial capacity up to 2.5%.

3.0 ANALYTICAL MODEL AND FORMULATION

3.1 Concrete Model of Jacketed Section

In concrete jacketing, a number of previous studies considers the weighted average of concrete strength (*fc'avg*) and some of the studies took old concrete strength (*fc'old*) for their analysis instead of combined strength. Some other studies considers smaller elastic modulus in their analysis for combined action. Generally, compressive strength is increased for the active confinement that can be determined by various equations derived by (Karayannis and Sirkelis 2008; Mander et al. 1988; Sheikh et al. 2002; Wu et al. 2006). In this regard, a concrete model is proposed to account *fc'old* which is expected to be increased with the confining stress generated by the jacket. Considering the confinement effect, thickness of jacket concrete in between longitudinal bar and old column face is only used to determine confining stress *fi'*. Finally jacketed compressive stress *f'cj* is determined using Mander concrete model (Mander et al. 1988).

3.2 Derived equations for compressive stress

Volumetric ratio of the new concrete and old column sections may be determined by the modification of FRP formula as derived by (Sheikh et al. 2002)

$$\rho_{con} = \frac{4 \times t_{jinn}}{b_0} \tag{1}$$

where,

t_{jinn}=thickness of concrete in between jacket and existing column section

b_o= least dimension of the existing column

According to Mander et al.(1988), confinement coefficient may be given by

$$k_e = \frac{1}{1-\rho_{con}} \tag{2}$$

Since the jacketed concrete is surrounding the total surface of existing column section, therefore clear spacing between lateral confinements are s'=0. Then s'/ds =0 where *ds*= diameter of lateral confinement. Rupture or tensile strength of concrete is generally presented by

$$f_{rc} = 0.7\sqrt{f_{c'new}} \tag{3}$$

where *fc'new* = compressive strength of jacketed concrete in MPa. Therefore, the confining stress

$$f_l' = 0.5 \times k_e \times \rho_{cc} \times f_{rc} \tag{4}$$

Volume of confined concrete to the volume of column section ratio for a jacketed column section can be presented by the following eqn

$$\rho_{cc} = \frac{(b_0 + 2t_{jinn}) \times (h_0 + t_{jinn})}{b_0 h_0} - 1 \tag{5}$$

ho = Larger dimension of column section

Finally, *fc'*j is given using Mander et al. (1988) confined concrete formula

$$f_{cj}' = f_{c'old}\left(-1.254 + 2.254\sqrt{1 + \frac{7.94 fl'}{f_c'_{old}}} - \frac{2fl'}{f_c'_{old}}\right) \tag{6}$$

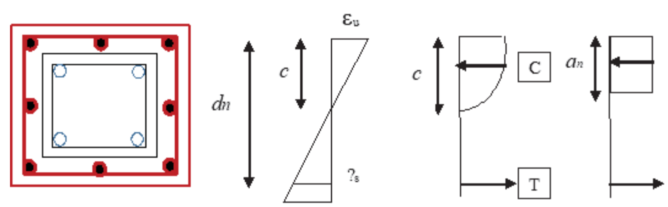

Fig 5: Column section subjected to lateral loading with strain compatibility

Columns are usually subjected to lateral loading due to wind force and seismic action. As presented in Fig.5, contribution of both new and old rebar can be accounted by elastic principle. Practically, in many cases old rebar are corroded and tensile strength is much lower than the new steel. In those cases the influence of old existing bar may be neglected for simplifications. In this study, strain compatibility is considered to derive the closed form equation for jacketed column as given in Eq. (7) and Eq. (8). Those equations can be readily used to depict simplified five point interaction diagram. Therefore, effective cross sectional area of concrete with intact rebar and jacketed area with rebar are taken into considerations. Using above stated equations, axial capacity of a jacketed column subjected to pure compression can be written as

$$P = CB \times [0.85\, f_{cj}' \times (A - A_{sold} - A_{sj})] + (A_{stj} + A_{scj}) \times f_{yj} \tag{7}$$

Where,

CB = Coefficient of bondage. The value of this

coefficient varies with different type of interface and bonding properties.
A= cross section after jacketing,
Asold = area of longitudinal steel in existing column,
Asj = area of longitudinal steel in jacket,
Astj = area of tension steel in jacket,
Ascj = area of compression steel in jacket,
fyj = yield strength of jacket longitudinal steel.

Old rebar bending capacity is considered to contribute along with jacketed bar. Since column remains in compression, old rebar bending capacity may not be fully utilized. Even if they reach to their yield stress due to continuous heavy lateral and earthquake loading for which they were not being designed, encaged jacketed rebar bending capacity will be increased by the remaining bending strength of rebar before it reach to ultimate stress. In overall bending capacity, to account for the remaining contribution of old longitudinal bar a partial value of their original bending capacity is assumed. It is denoted as Coefficient of moment (CM). It is considered that; these bar had reached to yield strength and their remaining ductility is added in the bending capacity of jacketed section.

Thus, $CM = 1 - \frac{f_{yold}}{f_{uold}}$; where, $\frac{f_{yold}}{f_{uold}}$ = is ratio of yield and ultimate strength of old column rebar. Generally, CM gives a value ranging from 0.20-0.35 depending upon their strength. Moment capacity of jacketed steel section may be written as

$$M = [A_{stj} \times f_{yj} + CM \times A_{stold} \times f_{yold}] \times (d_n - \frac{a_n}{2}) \quad (8)$$

Astold= area of tensile steel in old column, *dn*= distance of centroid of tensile jacketed steel from top fibre, a_n= dimension of equivalent stress block in jacketed section given as:

$$a_n = \frac{[A_{stj} \times f_{yj} + CM \times A_{stold} \times f_{yold}]}{0.85 f c'_j \times b_{new}} \quad (9)$$

Where,
b new = width of section after jacketing.

Jacketed concrete strain $\varepsilon_u j$ is taken as 0.003 in analysis. In contrary, old concrete is already stressed with induced strain. Thus old concrete strain is considered up to 0.0033 in this analytical equation. A single concrete strain is considered for simplification basing on strain compatibility. In doing so, weighted average basing on strain and force for both the concrete to is taken to account the average concrete strain in balance, compression and bending control points.

$$\varepsilon_{uavg} = \frac{A_{old} \times f_{c'old} \times \varepsilon_{uold} + A_j \times f_{c'new} \times \varepsilon_u j}{A \times f_{c'j}} \quad (10)$$

Where,
A_{old} , A and A_j =area of old, jacket and total section after jacketing respectively. Strain in reinforcement ε_s is taken as 0.0033 for bending control, $\varepsilon_s = \frac{f_{yj}}{E}$ for balance point and $\varepsilon_s = \frac{f_{yj}}{E \times 2}$ in compression control. From equivalent stress block of jacketed section,

$$c = \frac{\varepsilon_{uavg} \times d_n}{\varepsilon_s + \varepsilon_{uavg}} \quad (11)$$

and $a_n = \beta_1 \times c$ where value of coefficient β_1 is a function of $f_{c'j}$. Strain in steel in compression $\varepsilon_s' = \frac{\varepsilon_{uavg} \times (c - d_n)}{c}$ and $f_s' = E \times \varepsilon_s'$.

Finally following equations are proposed for balanced, compression and bending control points:

$$P = 0.85 f_c'_j \times a_n \times b_{new} + A_{sc\,j} \times f_s'_j + A_{stj} \times f_{yj} \quad (12)$$

$$M = 0.85 f_c'_j \times a_n \times b_{new} \times (\frac{h_{new}}{2} - \frac{a_n}{2}) + A_{scj} \times f_s'_j \times (\frac{h_{new}}{2} - d'_{new}) + A_{stj} \times f_{yj} \times (d_{new} - \frac{h_{new}}{2}) + CM \times A_{stold} \times f_{yold} \times (d_{old} - \frac{h_{old}}{2}) \quad (13)$$

4.0 FINITE ELEMENT MODELLING USING ETABS 2015 AND SAP 2000

This study also uses most commonly used building design software ETABS 2015 and SAP 2000 v17 to model jacketed column and hence to investigate the result in terms of column interaction diagram. In the process of modeling of jacketed column with the software, section designer was used to draw the exact shape of the column with new and old rebar position. For two different type of concrete area, separate concrete and material properties were also assigned. In such type of model, the program usually considers perfect bonding at the interface between new and old concrete. All other properties including longitudinal bar, tie bars and clear covers are defined according to the test samples. After complete modeling and material properties, stress contour in the section, moment vs. curvature and interaction diagram can be extracted as output from the software. SAP 2000 produce advance features for analysis stress and strain in different conditions as well as moment vs. curvature, steel strain and compression data. Typical cross section of jacketed column, moment curvature and strain at a certain point is presented in Fig.6. Interaction diagram of jacketed column is presented in result section to compare with analytical results and other codes.

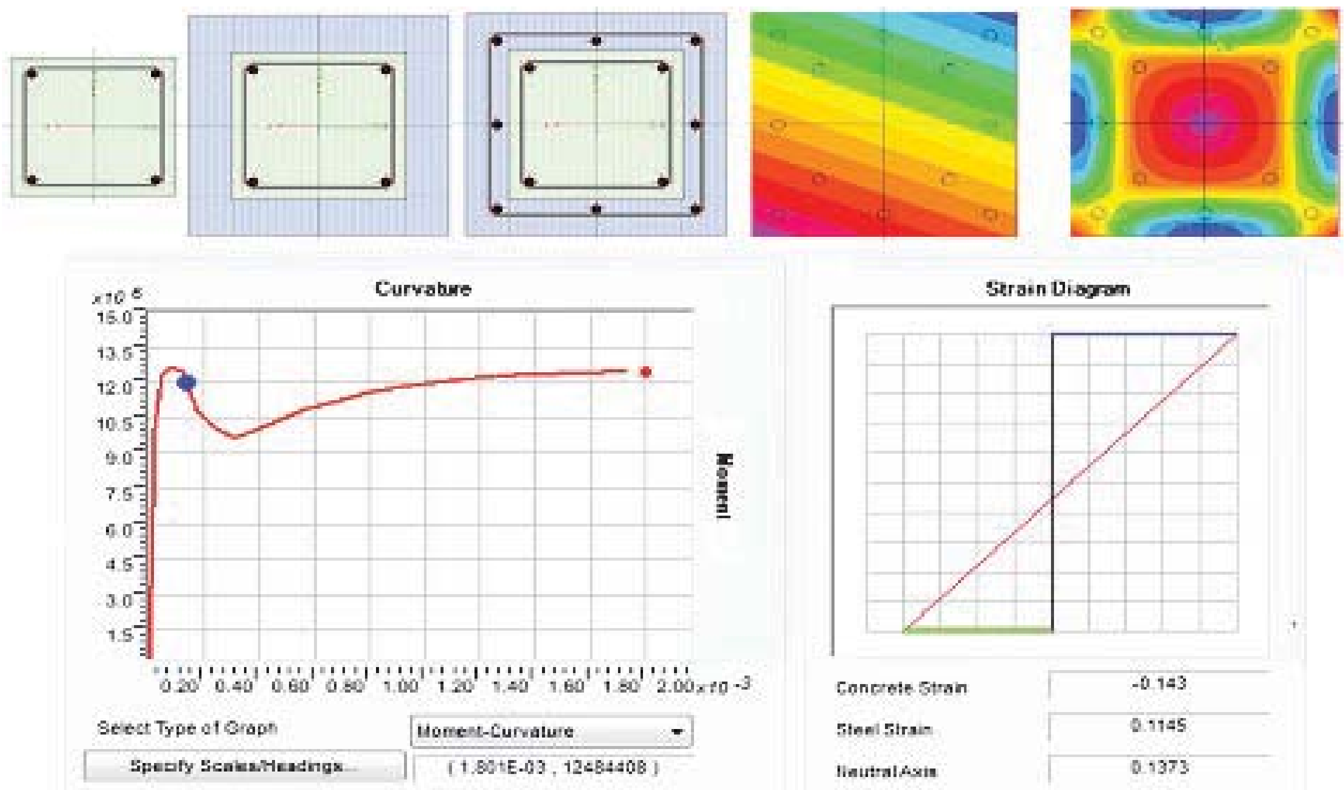

Fig 6. Jacket section in ETABS2015 and Sap2000 with moment curvature and typical strain diagram.

5.0 RESULTS COMPARISON

5.1 Experiment vs. Analytical Assessment

Using same geometrical properties, analytical capacity of the column samples are determined from the derived equations. The column capacity as obtained from Experimental results are compared with analytical results as presented in Table 2. Results presented in the table are the comparison of axial capacity of column. The samples are classified in two categories based on the area ratio 2.220 and 2.617. It is expected that the axial capacity of all short column directly varies with the cross sectional area. If it all samples are monolithically casted instead of jacketing, their strength ratio should be the same as area ration. However, the capacity ratio for the samples having area ration 2.220 varies from 1.52 to 2.06 subjected to their bonding type. On the other hand, samples having area ration 2.617 show a capacity ratio in between 2.1 to 2.30. It is clear from the samples of both area ratio that samples without having any bonding agent or surface treatment gives lower capacity after jacketing. In addition, bonding behavior does not follow a liner behavior and vary greatly according to construction, material property and types of surface preparation, moisture content of substrate [16]. The effect of creep and any direct tension stress caused by shrinkage are ignored. All these factors contribute in the deviations of results. Monolithic samples 25-M and 31.5-M had the less deviation due to absence of different concrete interface. 25-N and 31.5-N had the larger deviation due to the absence of bonding agent and surface preparation. Based on the analytical and experimental result, a set of values for coefficient of bonding, CB values are proposed as presented in Table 3.

5.2 Comparison of Axial and Moment capacity

Axial capacity of column samples as obtained from analytical formula and experimental result are compared with that obtained from ETABS2015 and SAP2000 v17 and Japanese code which is one of the existing retrofitting codes. The axial capacity comparison is presented in Fig.7. The figure clearly shows that the value obtained from ETABS 2015 and SAP2000 over estimates the capacity since they are unable to consider the interface effect such as surface treatment or application chemical bonding at the interface. In case of axial capacity, Japanese code also gives higher axial capacity but which is also lower than the commonly used software packages

Table 2: Comparison between Experimental and Analytical Results

Sample Name	*fc'$_{old}$ (MPa)*	*fc'$_{jacket}$ (MPa)*	*Area Ratio*	*Analytic capacity*	*Test capacity*	*Capacity ratio P_n/P_{old}*		*% variation*
			A/A_{old}	*(KN)*	*(KN)*	Analytic	Test	
25-N	11.12	19.35	2.220	243.29	226.21	1.522	1.521	7.02
25-B	11.13	21.63		289.64	258.51	1.948	1.739	10.74
25-B-W		21.98		289.75	264.38	1.813	1.778	8.78
25-M		21.80		289.69	302.5	1.812	2.030	4.36
25-B-C		21.89		329.83	298.83	2.064	2.010	9.46
31.5-N	11.12	24.98	2.617	336.9	305.67	2.108	2.056	9.26
31.5-B	12.20	29.42		441.00	416.78	2.759	2.803	5.51
31.5-M		29.87		441.72	452.86	2.764	3.046	2.69
31.5-B 2	11.12	20.85		403.70	379.45	2.526	2.552	6.01
31.5-B-W	11.12	20.21		402.43	382.78	2.518	2.575	4.88
31.5-B-C		21.32		372.08	342.68	2.328	2.305	7.74
RS		-		159.80	148.65			6.97

Table 3: Value of Coefficient of Bonding with interface properties

Surface Properties	Value of CB	Remarks
1. Non-bonded (No Surface treatment)	0.65	Sample 25-N and 31.5-N
2. Bonding and Surface Treatment	0.85	Chemical at interface
3. Bonding, Surface treatment and welded tie	0.88	Sample 25-B-W and 31.5-B-W
4. Monolithic Casting	1.00	Perfect Bonding

Moment carrying capacity of column samples as obtained from analytical formula are compared with that obtained from ETABS2015 and SAP2000 v17 and Japanese code as shown in Fig 8. Similar to the axial capacity comparison, both analysis tools give higher value of moment capacity than that found in this analytical study. The figure clearly shows that the value obtained from ETABS 2015 and SAP2000 over estimates the moment capacity than that obtained by both analytical result and Japanese code. This happens due to the fact that they are unable to consider the interface effect such as surface treatment or application chemical bonding at the interface. This happens due to the fact that in Japanese code considers uniform strain all through the length of the column and combined failure occurs at the interface. However, concrete having different strength and age is expected to deform differentially which allows less capacity than expected combined capacity.

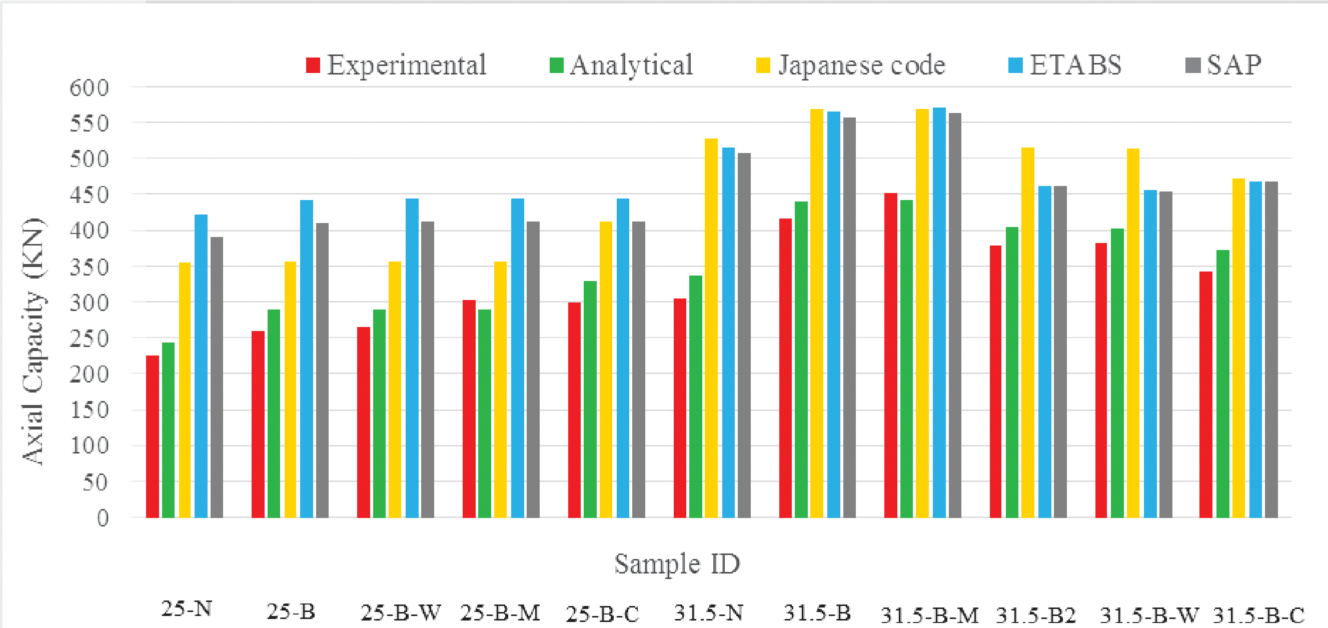

Fig 7: Axial Capacity Comparison

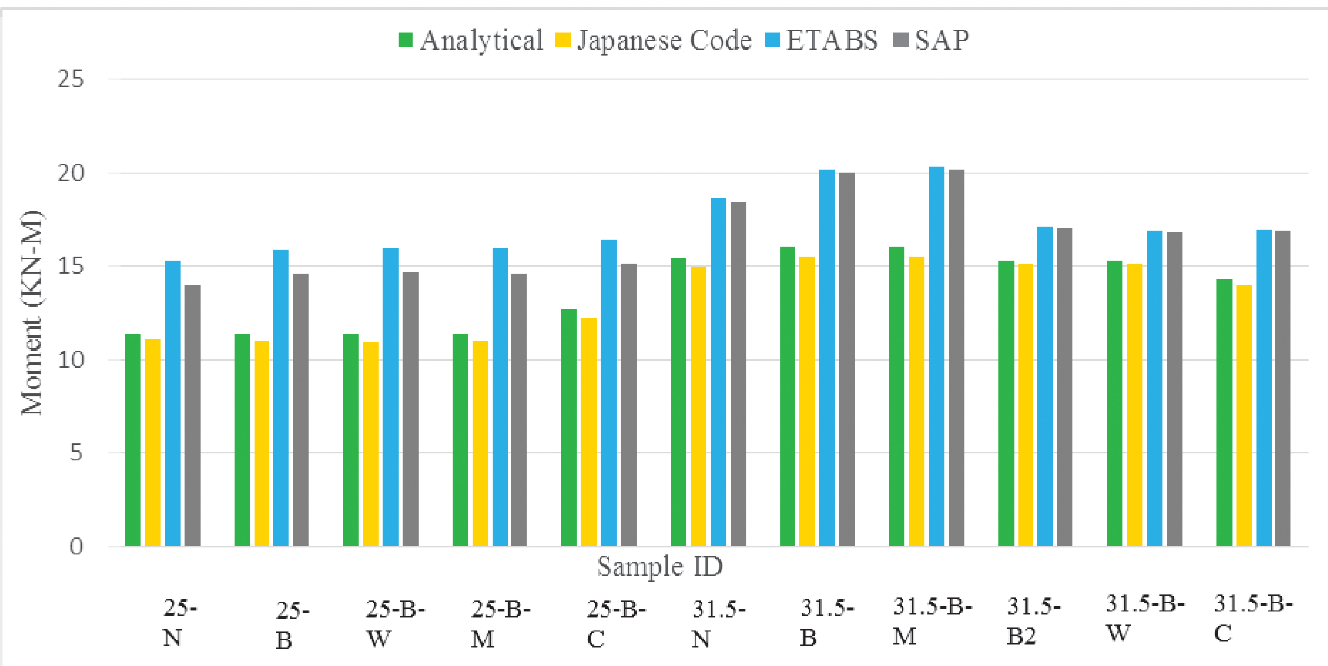

Fig 8: Maximum Moment Capacity Comparison

5.3 Comparison of Column Interaction

Finally results are compared in terms of column interaction diagrams as shown in Fig.9. In building the diagram, formulated equations are used and compared with Japanese code, finite element package ETABS and SAP2000. The horizontal and vertical axis of the diagram presents the axial and moment carrying capacity, respectively for a retrofitted column section. It can be concluded from the Figure that maximum axial capacity as derived in this study is as much as 26% lower than the methods and code. This happens due to the fact that all other procedures consider monolithic action at the new and old

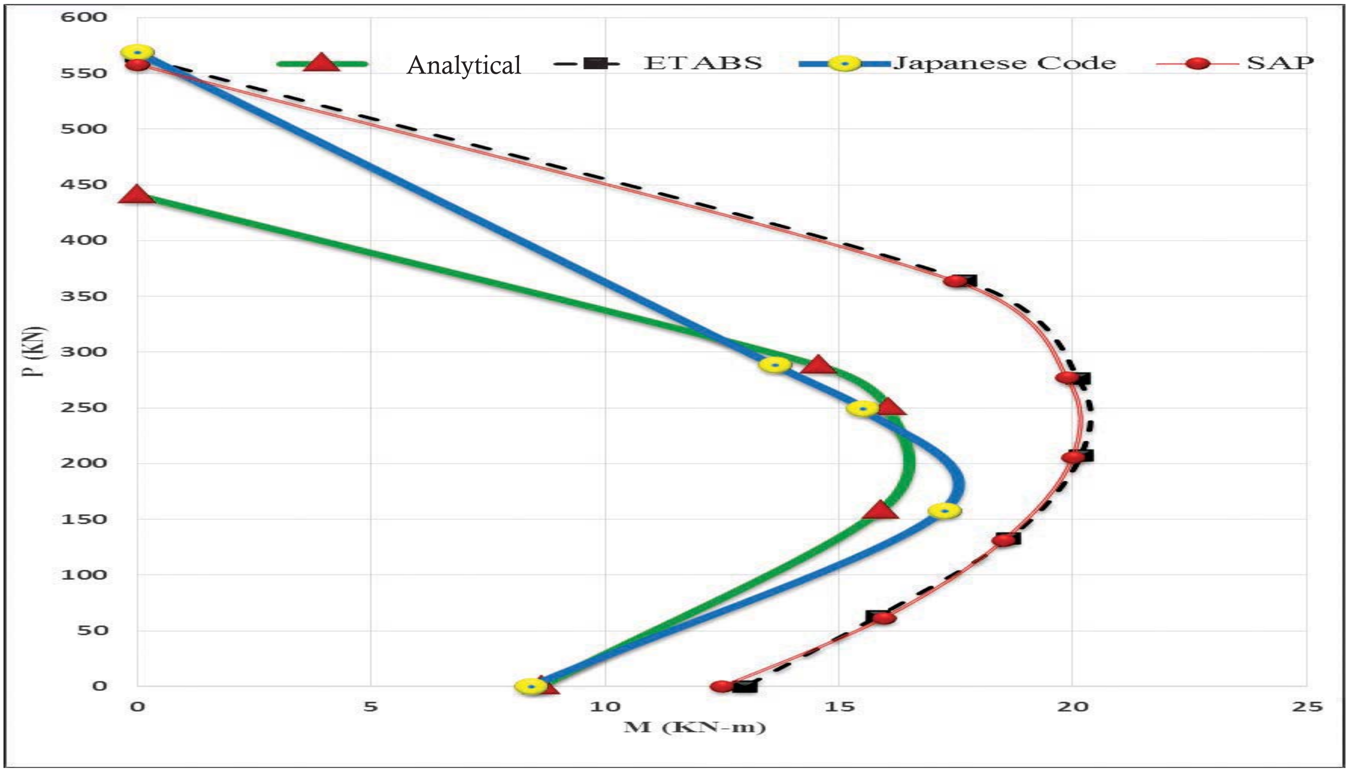

Fig 9. Comparison of Interaction Diagram

interface where this experimental and theoretical study suggest a reduction of capacity due to different interface.

In the comparison it is also clear that derived equation gives a close prediction of moment capacity with Japanese code in the balanced condition which is significantly lower than both SAP2000 and ETABS. In another word, proposed analysis agrees well with an accuracy of 95-98 % with the Japanese code particularly for maximum bending capacity which occurred mainly at the balanced point of interaction curve. However, both differ with ETABS and SAP with a deviation of 16.5-25% due to liner addition and composite action account which is shown in Fig. 7.

6.0 CONCLUSION

This paper investigates the structural capacity enhancement of column by RC Jacketing. A total no of twelve RC columns in one third scale has been tested to determine the actual capacity enhancement ability of jacketing technique with different interfaces. Experimental results reveals that RC column jacketing significantly increases the column capacity that varies interface to interface. It is also observed that column capacity or capacity enhancement depends on the material properties, jacket thickness and more importantly the interface type between new and old concrete. The failure pattern indicates that the outer jacket fails earlier than the core mainly due to the inability to maintain strain compatibility between new and old concrete. Closed formed analytical formulas are also proposed to estimate the axial and moment capacity of jacketed column section. Based on the interface type, a set value for coefficient of bonding (CB) is proposed in the range of 0.65 to 0.88. Lower value of CB is proposed for no surface treatment where higher value of CB represents a surface that ensures mechanical interlocking, chemical bonding and welded tie as well. After performing, experimental and analytical investigation, results are compared with finite element building software SAP2000, ETABS 2015 and Japanese code. The comparison shows that proposed study modifies the load carrying capacity that underestimates the axial capacity with all other procedure. On the other hand, moment capacity by proposed equation is close to Japanese code but nearly 20% less than that estimated by SAP2000 or ETABS2015. Therefore it is to be mentioned that the widely acknowledge building software such as SAP2000 and ETABS 2015 overestimates the jacketed column capacity that may lead to inaccuracies in designing real life projects. This research work may contribute to develop a design guideline for column strengthening using RC jacketing.

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