Large-scale Cyclic Loading Test on a Steel and Reinforcement Composite Bridge Pier Constructed by Automated Method

H.H. Hung, K.Y. Liu & P.H. Wang
National Center for Research on Earthquake Engineering, Taipei, Taiwan

Y.C. Sung
National Taipei University of Technology & NCREE, Taipei, Taiwan

K.C. Chang
National Taiwan University & NCREE, Taipei, Taiwan

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SUMMARY:
Reinforced concrete is the most widely used construction material for bridge piers in Taiwan. Due to the requirements of strength and ductility for seismic design of RC structures, a large number of reinforcement are usually required. This tight arrangement of reinforcement not only complicates the construction works but also deteriorates the quality of concrete casting. In order to solve this problem, a steel and reinforcement composite bridge pier system was proposed. Large scale experimental studies for the proposed system as well as the conventional system which was constructed based on the conventional construction technology were conducted. By comparing the experimental results and construction practices of the developed system with those of the conventionally detailed one, the seismic performance and the constructability of the proposed pier system are proved to be better than that of the conventional RC pier system.

Keywords: cyclic loading test steel and reinforcement composite piers automated methods in construction

1. INTRODUCTION

Reinforced concrete is the most widely used construction material for bridge piers in Taiwan. When the bridge pier is high, due to the requirements of strength and ductility in seismic design of RC structures, a large number of reinforcement, including longitudinal reinforcement, transverse reinforcement and internal cross ties, are usually required. The process for large amount of reinforcing binding is heavily reliant on skilled labors, which is time-consuming and costly. In addition, this tight arrangement of reinforcement not only complicates the construction works but also deteriorates the quality of concrete casting. Thus, for such a conventional bridge pier system based on the conventional construction technology, the construction period is highly likely to be long and the seismic performance of the pier is also likely to be inferior to that was expected. In order to solve this problem and also to improve the construction safety based on a reasonable construction cost, the purpose of this study is to develop innovative bridge pier systems which are based on automated methods in construction and also have good seismic performance. Two bridge pier systems which possess these features were proposed. One is the steel and reinforcement composite bridge pier system and the other is the multi-spiral stirrup pier system. In order to verify the constructability of the proposed method and to investigate the seismic performance of the proposed systems, large scale specimens for both systems as well as a conventionally detailed system were all constructed at the National Center for Research on Earthquake Engineering (NCREE) in Taiwan, followed by a cyclic loading test performed on each specimen. During the construction practice, not only was every construction steps carefully recorded and photographed, the time and manpower required for each step were also closely recorded. In this paper, focus is on the steel and reinforcement composite bridge pier system. By comparing the experimental results of the developed system with those of the conventionally detailed one, the seismic behavior of the proposed bridge pier was examined. Through the real construction practice at the laboratory and the comparison of the construction period and construction cost of each specimen column, the efficiency of the proposed construction method was identified and the construction cost was discussed.

2. SPECIMEN DESIGN

With an aim to improve the safety and efficiency of the bridge pier construction in Taiwan, a steel and reinforcement composite pier system which is suitable for the construction environment in Taiwan was proposed in current study with reference to the 3H method (Kunikazu et al. 1999 and Michio et al. 1998) which has been used in Japan. The proposed composite pier consists of several large H sectional steels inside and several vertical reinforcing bars outside. By replacing partial vertical reinforcement of the conventional RC pier by the H-shaped sectional steel, the number of the main reinforcement can be reduced, and the sectional steel can also provide the support for the stirrup cage and to hold the longitudinal bars in position. The intermediate hoop ties that are usually adopted in a conventionally RC pier are replaced with the stirrup cage which is pre-assembled by several so called one-bar hoops. Each of the one-bar hoops is formed with a single steel bar and closed at both ends by two hooks. The confinement effect of such type of hoop has been approved to be better than that of the conventional rectilinear hoops and stirrups (Chang et a., 2003). Thus, the pre-assembled stirrup cage can not only increase the efficiency of construction, the use of one-bar hoop can also enhance the confinement of the concrete core. In addition, because the composite pier consists of both steel and concrete, high shear strength and ductility can be provided by inner steel and the concrete around steel section can protect the steel from buckling.

In order to realize the seismic resistance of current proposed pier system as compared to the one with the conventional design details, a conventional RC specimen column which was designed based on current seismic design code in Taiwan (MOTC, 2008) was also constructed. Thus, a total of two 1/2 scaled specimens were designed and constructed at NCREE. One is a RC column with the conventional design details, and the other is the proposed steel and reinforcement composite column. The target pier for the test specimen is a rectangular bridge pier with a height of 18 m and a reinforcement ratio of 1.5%. Thus the scaled specimens are 9 m in clear height with a cross section of 1.8mx1.2m, as shown in Fig. 1(a). The design details of both specimens are also schematically shown in Fig. 1 (b) and (c).

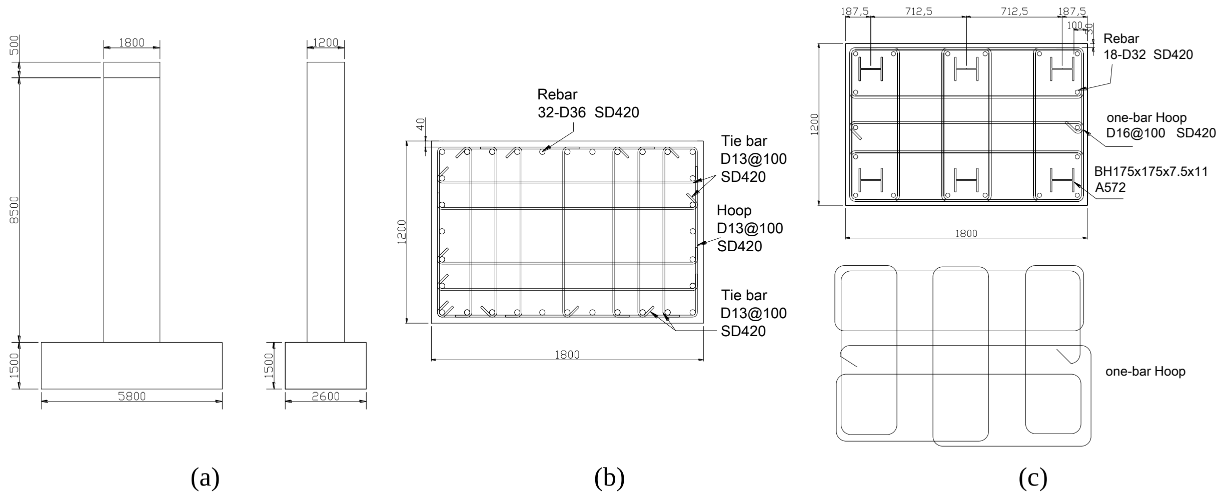
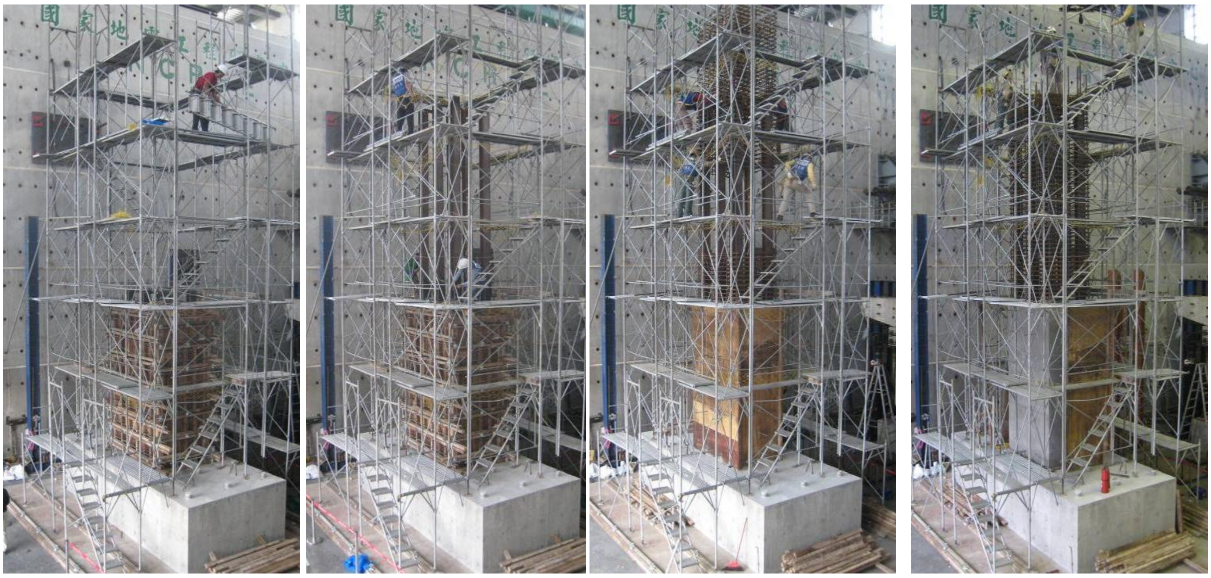


Figure 1. Design details of specimen (a) side view of the specimens (b) design details of the conventional column (c) design details of the proposed composite column

As can be observed in Fig. 1, the specimen with the conventional details was reinforced with 32-D36 SD420 rebars (steel ratio = 1.5%) and transversely reinforced with D13 perimeter hoops and internal stirrups spaced 10 cm (volumetric confinement ratio = 1.19%). The steel and reinforcement composite column was reinforced with 18-D32 rebars and six A572 RH 175×175×7.5×12 sectional steel. By such an arrangement, the steel ratio of the composite specimen is 0.68% for the rebar and 1.39% for the sectional steel. The total steel ratio with respect to the rebar is 1.83%, which is larger than the conventionally detailed one. The conservative design of the composite column with a large steel ratio is because the bonding effect between the H sectional steel and the surrounding concrete still needs to be verified through the test, and the effective depth of the rebars for the composite column is a little bit smaller than that of the benchmark conventional specimen. In addition, due to the arrangement of the H sectional steel, the composite column was transversely reinforced with D16 one-bar hoop as shown in Fig.1 (c). To have the same pitch as the conventional column, i.e., 10 cm, the volumetric confinement ratio of the proposed column becomes 1.40%, also larger than that of the conventional column. To achieve a good bonding effect between the steel and the surrounding concrete, four shear studs were also welded on the surface of each H sectional steel for every 20 cm in the potential plastic region. The nominated material properties for these specimens are as follows: concrete compressive strength is 350 kg/cm^2; yield strength of both main reinforcement and transverse reinforcement is 4200 kg/cm^2; the yield strength of the sectional steel is 3500 kg/cm^2.

3. CONSTRUCTION PROCESS

In a real construction practice, it is unavoidable that the vertical reinforcing bars and the steel sections have to be separated into several parts. Thus, the quality control of the connection between the separated rebars and steel sections becomes a crucial issue for seismic performance and a key factor for the efficiency of construction. In order to simulate the construction of the connections in our construction practice, all the vertical reinforcing bars and H-shaped sectional steel were separated into two sections with a connection at the height of around 4 m above the foundation. Thread Couplers were adopted to join the rebars and bolts were used to connect the H-shaped sectional steel.



(a) (b) (c) (d)

Figure 2. Construction photos for the proposed composite column (a) erection of scaffold (b) erection and connection of the steel section and part of the main bars; (c) installation of the pre-assembled hoop cage; (d) erection and connection of the remaining main bars.

For the construction of the proposed composite column, the first step is the erection of the prefabricated segment of steel sections. The second step is the erection of the pre-assembly hoop cage and the installation of the main bars, followed by the binding of the foundation reinforcement. The third step is the setup of the formwork, and then the concrete are poured into the lower part of the column. For the upper part of the column, the first step is the erection of scaffold, followed by the erection of the steel section. The steel sections are erected one by one and are connected to the lower part of the steel sections by bolts. Then parts of the main bars which are not likely to interfere with the installation of the hoop cage are erected at first, followed by the installation of the pre-assembled hoop cage. The next step is to erect the remaining main bars and connect them to the lower part of the rebars by couplers. Then after the setup of the form work, followed by the pouring of the concrete, the construction is completed. In order to demonstrate the construction sequence of the proposed composite column, the construction photos are given in Fig. 2. For brevity, only the construction photos for the upper section are listed. In Fig. 2, (a) shows the erection of the scaffold; (b) shows the erection and connection of the H sectional steel and part of the main bars; (c) shows the installation of the pre-assembled hoop cage, and (d) shows the erection and connection of the remaining main bars. By comparing these construction photos with those of the conventionally RC column as given in Fig. 3, no tedious and complicated on-site binding process of reinforcement is needed for the proposed composite column. So the quality and efficiency of the construction can be improved.

The construction method for the specimen was developed basically by effectively combining existing technologies. To further enhance the construction efficiency of the proposed composite column, the main reinforcing bars can be pre-assembled with the hoop cage with another type of couplers, which doesn't need to rotate the rebars when joining two separated rebars, in order to prevent the installation of the main bars on-site. To ensure a good efficiency and safety of the construction, there are also some matters need to be noted. For instance, (1) the initial assembly of the hoop cage has to be performed precisely. (2) The length of the pre-assembled cage must be as long as possible. (3) Depending on the bridge pier height and the construction conditions, appropriate measures must be taken to prevent vibration, oscillation of the steel section during erection.

(a) (b) (c) (d)

Figure 3. Construction photos for the conventionally detailed column (a) erection of the lower main bars; (b) binding of the reinforcing bars for the lower part; (c) erection and connection of the upper main bars; (d) binding of the reinforcing bars for the upper part

4. EXPERIMENTAL PROGRAM

In order to investigate the seismic performance of the proposed steel and reinforcement composite column which was constructed according to the proposed constructed procedure, cyclic loading test were conducted on two specimens, including one designed and constructed through the conventional way, at NCREE. A short summary of the experiments is provided here, including the experimental setup, the instrumentation arrangement and the loading protocol.

Fig. 4 illustrates the test setup. Sixteen high tensile strength tie-down rods with a diameter of 69 mm were placed through the footing and anchored into the strong floor of the laboratory to simulate the fixed-base condition of the foundation. During the test, an axial load of 5186 kN was applied to the test column through a tap beam using two vertical high tensile strength rods. The vertical loading was kept constant throughout the test to simulate the tributary dead load of the deck, which is around $0.07A_gf_c'$. In which, A_g is the gross cross-sectional area of the column. In addition, three horizontal actuators were used to apply the lateral force to the column's top to simulate the seismic loading. The location of the application force was 8.5 m up from the top of the footing.

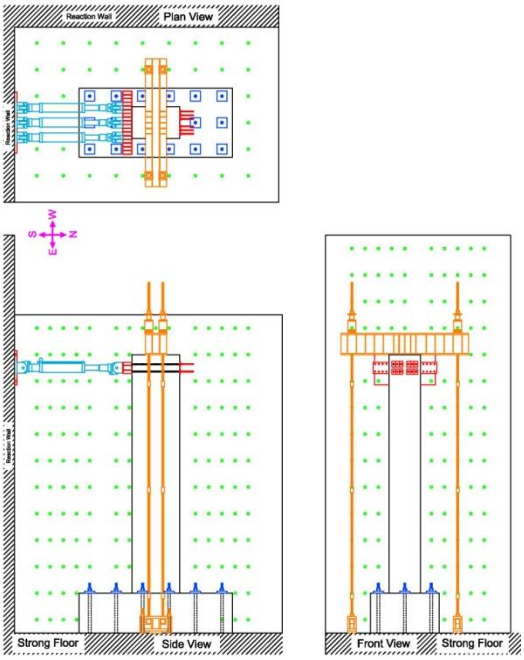


Figure 4. Schematics of experimental setups

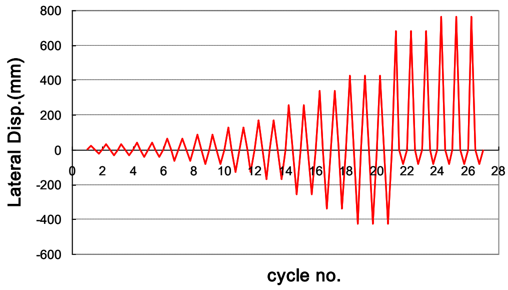
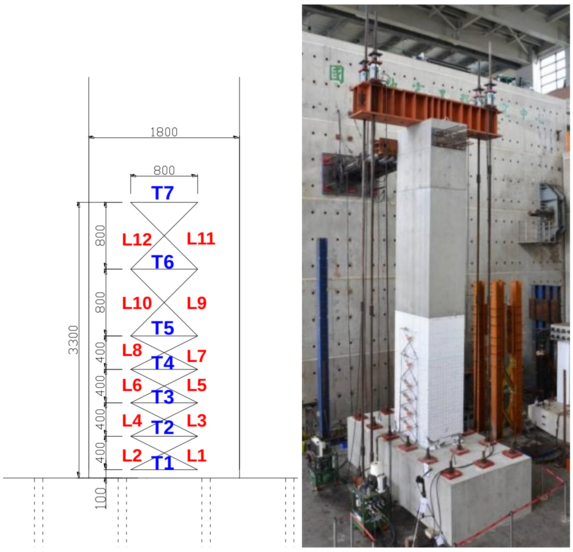
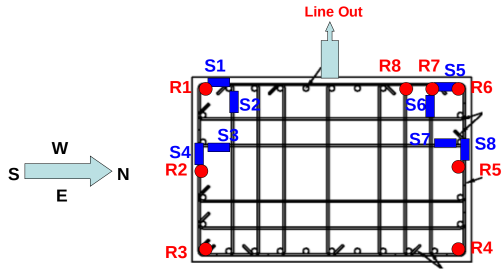


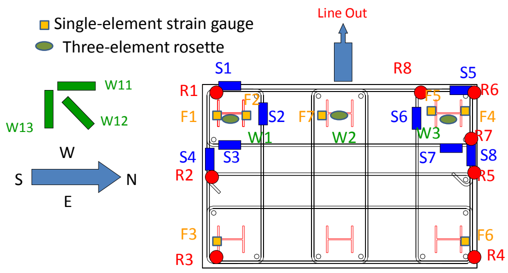
Figure 5. Loading protocol for the cyclic loading test



(a)



(b)



(c)

Figure 6. Instrumentation arrangement (a) tiltmeter and LVDT displacement gauge; (b) arrangement of strain gauge for the conventionally detailed columns; (c) arrangement of strain gauge for the composite column

Displacement-controlled cyclic loading test was performed on these two specimens. Fig.5 shows the displacement loading protocol for the test, where the excited drift ratios include 0.25%, 0.375%, 0.5%, 1.0%, 1.5%, 2.0%, 3.0%, 4.0% and 5.0%. The prescribed displacements were applied on the column two cycles for each drift ratio which is equal to or lower than 4%. For the drift ratio other than these values, the corresponding lateral displacement was applied on the column top for 3 cycles. In addition, considering that the composite column may have a better ductility than the conventionally detailed one, drift ratios larger than 5%, i.e., 8% and 9%, were also applied. However, due to the stroke limit of the actuators, the drift ratios 8% and 9% were only applied along the North (push) direction; while along the South (pull) direction, the applied drift ratios were only 1%.

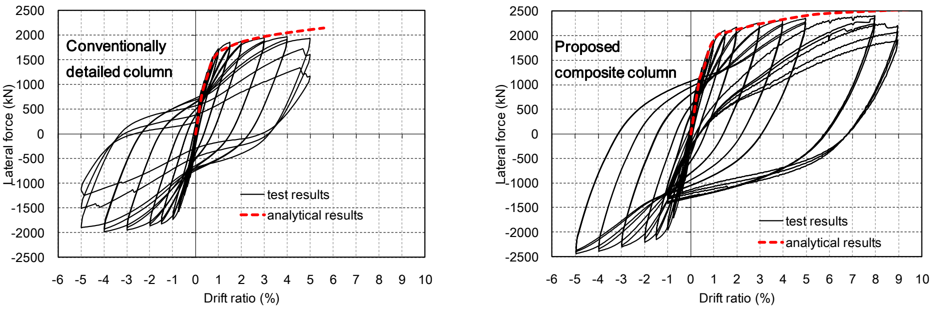
In order to measure the curvature and shear displacement of the test columns under the excitation of cyclic loadings, seven tiltmeters and twelve LVDT displacement gauges were mounted on the east side of the specimens as shown in Fig.6 (a). Tiltmeters T1 to T7 were mounted at distances of 10cm, 50cm, 90cm, 130cm, 170cm, 150cm and 330cm above the foundation top. Displacement gauges L1~L12 were crossly mounted between the tiltmeters. In order to measure strain of the rebars and the sectional steels, several strain gauges were installed on the suitable location of both specimens. Fig. 6 (b) and (c) schematically show the layout of the strain gauges for the conventionally detailed specimen and the composite specimen, respectively. Symbol *R* represents the strain gauge on the main rebars and was installed at the cross section 10cm, 160cm and 318cm above the foundation. Symbol *S* represents the strain gauge on the transverse reinforcement and was installed at a location 15cm, 154cm and 316cm above the foundation. For the composite specimen, symbols *F* and *W* represent the single-element strain gauge and the three-element rosette which were mounted on the steel section at a distance 10cm, 160cm and 318cm above the foundation.

5. TEST RESULTS

Fig. 7 shows the hysteretic curves for the test columns under the excitation of the cyclic loading, where (a) and (b) represent the results for the conventionally detailed specimen and the composite specimen, respectively. As can be seen, the lateral strength for the conventionally detailed one is around 2000 kN, and the strength degraded significantly at the second cycle of the displacement corresponding to the drift ratio of 5%. In addition, the strength continued to reduce to a low value of 1200 kN after the third cycle of 5% drift ratio. Thus the test ended at this moment. As for the composite specimen, the lateral strength of the specimen is around 2400 kN and the strength did not degrade after the third cycle of drift ratio 5%. Thus, a drift ratio of 8% along the push direction and 1% along the pull direction were continuously conducted on the column. After the third cycle of the displacement corresponding to the drift ratio of 8 %, the lateral strength of the column still did not degrade significantly, so 9% drift ratio excitation was performed continuously. The lateral strength degraded to 1800kN after the third cycle of 9% drift ratio, which is lower than 80% of the specimen’s maximum strength, so the test ended. To sum up, both the lateral strength and ductility of the proposed composite column is higher than those of the conventional one. Another difference between these two columns observed in Fig. 7 is that the strength of the composite column continued to increase after the lateral force reached its nominal strength, whereas the strength of the conventional column remained almost a constant value after the nominal strength was reached. This is because for the composite column, once the cracks in concrete occurs and result in the reduction of stiffness, the steel section can still provide shear capacity and ductility to resist the subsequent cycles of overload. Therefore the strength can continue to increase after the column reaches its inelastic state.

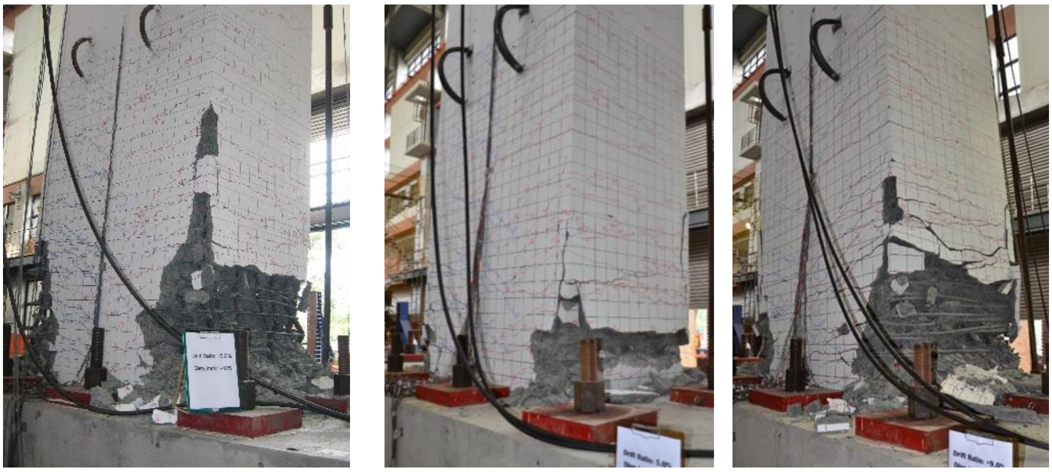
The failure photos for the specimens are given in Fig. 8, where (a) and (b) show the photos of the conventionally detailed column and the composite columns, respectively, after the excitation of drift ratio 5%, and (c) shows the failure photo of the composite column after the excitation of drift ratio 9%. By comparing Figs (a) and (b) at the same drift ratio of 5%, the superior ductility of the composite column can be clearly observed. Several main reinforcing bars were buckled and fractured at the drift ratio of 5% for the conventional columns, whereas for the composite column at the same drift ratio of 5%, the transverse reinforcement was exposed, but not the main reinforcing bars. Therefore, the lateral strength for the composite column did not degrade at this stage. After the third cycles of the drift ratio 9%, some vertical reinforcement of the composite column buckled and fractures as shown in Fig. 8 (c). Thus, the lateral strength declines to around 1800kN. In addition, the superior confinement effect of the one-bar hoop used in the composite column can also be clearly observed in the close-up photos given in Fig. 9, where figures (a) and (b) show the failure photo of the conventional column after the excitation of 5% drift ratio and the failure photo of the composite column after the excitation of 9% drift ratio. As can be seen, even though the composite column was subjected to a higher value of drift ratio than the conventional one, most of the rectilinear hoop was still in its original position and provided the confinement for the core concrete.

Fig. 10 (a) and (b) show the vertical distribution of the curvature in the potential plastic hinge region for each test column. The average curvature was obtained by taking the difference between the readings of two adjacent tiltmeters divided by the distance between them. It shows that failure was localized at the bottom of the column for both specimens, especially for the composite column. The same phenomenon can also be observed from the photo in figure 8(b). However, the curvature for the area outside the lower 30 cm region of the composite column was smaller than that of the conventionally detailed column. This means that the damage occurred on the composite column was minor as compared to the conventional one. This phenomenon can be attributed to the superior confinement effect of the one-bar hoop adopted in the composite column.



(a) (b)

Figure 7. Experimental results (a) conventionally detailed column (b) proposed composite columns



(a) (b) (c)

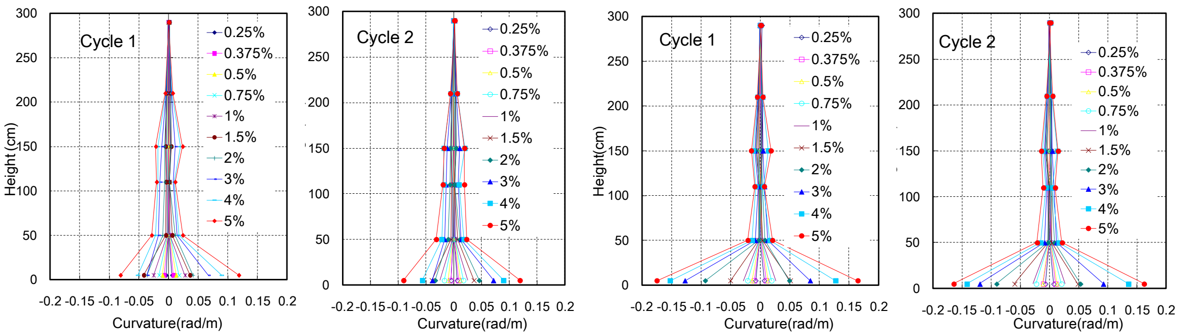
Figure 8. Failure photos for (a) the conventional pier after drift ratio 5% (b) the proposed pier after drift ratio 5% (c) the proposed pier after drift ratio 9%

Fig. 11 shows the comparison of shear displacements between these two specimens for the second cycle of the cyclic loading. In which, the value of the vertical coordinates represents the percentage of the shear displacement with respect to the total displacement, and figures (a) and (b) represent the result for the loading applied along the pull (south) direction and the push (north) direction, respectively. As can be observed in Fig. 11, the composite column has a much lower value of the shear displacement than the conventional column. This result implies that shear force resistance of the composite column is better than that of the conventionally detailed one.

As mentioned previously, the design of the composite column was very conservative due to the unknown of the bonding effect between the H-sectional steel and the surrounding concrete. After the test, however, the assumption that the steel section and the surrounding concrete are fully bonded with each other can be confirmed by comparing the test results with the analytical results which is calculated based on the assumption of a fully composite column. In Fig. 7, very good consistency can be observed between the analytical results and the envelop of the test results. This observation implies that the steel and the surrounding soil can be assumed to be fully composite and the original design for the composite column is too conservative. Based on the assumption of a fully composite column, the design moment strength for both columns were re-calculated. The design moment strength M_n (as the strain of concrete reaches 0.003) for the conventionally detailed column and the composite column are 14199 kN-m and 16190 kN-m, respectively. By dividing the lateral forces shown in Fig. 7 by the design shear strength V_n corresponding to the design moment strength (V_n = M_n/8.5m), the normalized hysteretic curves for both columns can be obtained and given in Fig. 12. As can be seen, the lateral strengths for these two specimens after normalization are almost the same. This result indicates that if these two specimens were designed based on the same design moment strength, their observed strength during the test should be the same, too.

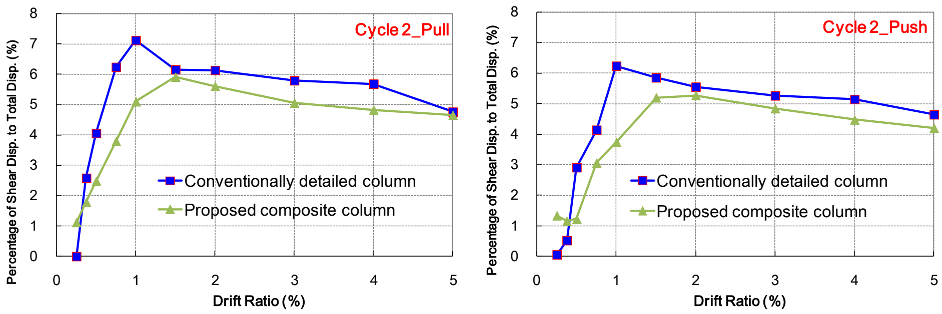
South side North side South side North side
(a) (b)

Figure 9. comparison of failure mode (a) conventional pier for drift ratio 5% (b) proposed pier for drift ratio 9%



(a) (b)

Figure 10. Curvature distributions for different drift ratios (a) conventional pier (b) proposed pier



(a) (b)

Figure 11. Percentage of shear displacement to the total displacement for the second cycle of cyclic loading which was applied along: (a) the south direction (pull); (b) the north direction (push)

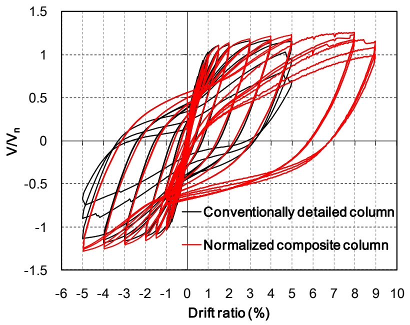


Figure 12. Comparison results after normalization.

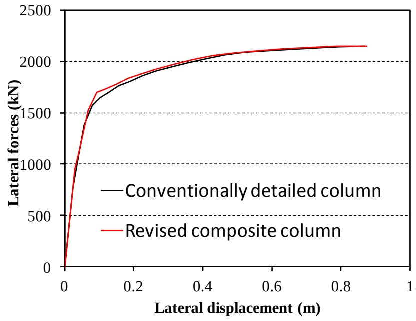


Figure 13. Comparison of the analytical results

For the selection of a practical pier system, not only the seismic performance and constructability are the important issues, the construction cost is also a decisive factor. Therefore, besides investigating the seismic performance and constructability of the proposed column system, the construction cost of such a column system was also evaluated and compared to the conventional one. In order to let the two comparative columns have the same comparative base, i.e., the same design strength, a new composite column which has the same design strength as the conventionally detailed column was designed. This revised composite column has the same amount of main rebars as the original one, but the original RH 175×175×7.5×12 sectional steel is replaced by RH150×150×7×10. By so setting, the steel ratio becomes 0.68% for the rebars and 1.09% for the sectional steels. The total steel ratio with respect to the rebar is 1.59%. In addition, the pitch of the transverse reinforcement is increased to 11.7cm in order to have the same volumetric confinement ratio as the conventional column, i.e., 1.19%. Based on this new design, the lateral force vs. lateral displacement curve for the composite column was calculated and compared with the conventional one in Fig. 13. This figure indicates that the newly designed composite column has the same strength as the conventional column. Therefore, the comparison study for the construction cost in the next section will be based on this revised design.

6. COMPARISON AND DISCUSSION

As has been mentioned previously, constructability, construction cost and seismic resistance are all the crucial issues for the selection of a practical pier system. Therefore, in this section, the constructability and the cost of the proposed columns were discussed through the comparison with the conventionally detailed one. The seismic performance of the proposed system was also summarized.

The construction work rate is the direct indicator for the efficiency of construction, so the constructability of the proposed column was compared with the conventionally detailed one in the form of construction work rate and given in Table 1. Construction work rate, which is the product of the number of labour worker and the working time, represents the total amount of uninterrupted labour required to perform a task. As can be observed in Table 1, the construction work rate for the composite column is 381 man-hours, which is lower than that of the conventionally detailed one, i.e., 407 man-hours. This information confirms that the construction efficiency of the proposed column is better than that of the conventional one, and therefore the proposed system can be adopted to overcome the problem of the shortage of skilled worker in the labour market. Table 2 shows the revised cost of specimens, where the cost of the composite column represents the estimated cost of the newly designed composite column which has the same strength as the conventionally detailed column. As expected, the cost for the composite specimen is higher than that for the conventional one, i.e., 129% of the conventional one. This can be attributed to the high cost of steel and more overlapping transverse rebar for the usage of the one-bar hoop.

In summary, both constructability and seismic performance of the proposed composite column are better than those of the conventionally detailed one. However, for such a short test column with a low steel ratio constructed in current study, the construction cost of the composite column is higher than that of the conventionally detailed one. Consequently, the promotion of the proposed system for its usage in typical bridge piers may not be easy currently. Nevertheless, for a high pier with a high steel ratio, the difference of cost between the composite column and the conventional RC column is expected to be smaller and the advantage of the proposed composite column can be revealed. In addition, for a pier with a high steel ratio, it is difficult to arrange all the needed vertical reinforcing bars in a conventional RC column, and the replacement of partial rebars with the steel section become a solution.

Table 1. Comparison of construction work rate (unit men-hour)

item	Conventional pier	Composite pier
Foundation	152.1	146
Lower part of the pier	86	72
Middle part of the pier	106.8	100.5
Upper part of the pier	61.9	61.9
total	406.8	380.4
%	100%	93.5%

Table 2. Comparison of the construction cost for the specimens (unit: NT dollar)

item	Conventional pier				Composite pier (revised one)			
	processing charges	Construction charges	Material charges	total	processing charges	Construction charges	Material charges	total
foundation	5,795	57,398	20,4021	267,214	5795	57398	204021	267214
column	4,079	119,727	263,691	387,498	146,997	102,825	327,134	576,956
total				654,711				844,170
percentage				100%				129%

7. CONCLUSIONS

This paper proposes a steel and reinforcement composite bridge pier system. The most tedious and complicated work for the conventionally detailed bridge pier is the binging of the reinforcing bars. Owing to the automation work of the pre-assembled hoop cage and the support provided by the steel section, the construction efficiency of the proposed system was proved to be better than that of the conventional one through a construction practice performed in this study. Thus, this method can be adopted to overcome the problems of skilled labour shortages. In addition, from the cyclic loading test, it can be concluded that the seismic performance of the proposed composite column can not only reach the standard for the conventional RC column, its ductility can be even better than that of the conventional one. However, the total cost of the proposed composite specimen is higher than that of the conventionally detailed one due to the high cost of steel.

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