

# Static cyclic-loading test on new type of rigid connection of steel girder and reinforced concrete pier

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**Abstract:** A new type of rigid connection of steel girder and reinforced concrete pier of a bridge is proposed. The components in rigid connection are installed by high-strength bolts on the spot, which are very convenient in construction. The moment from superstructure can be effectively transferred to substructure, and the plates provided for shear transferring can withstand the majority of total horizontal force. With static cyclic-loading test, useful experimental data is obtained on the new type of connection of steel superstructure and concrete substructure. As a result, the stress transfer mechanism of the rigid connection can be made clearly and the seismic performance of this structure can also be clarified. Compared computed strength and ductility with actual results, it can be found that this type of connection has good energy absorption capacity in spite of large displacement and no local buckling arises at the locations where stress concentration occurs. Because of doing away with the expensive bearing, this new type of composite structure can be expected to construct a bridge with high seismic resistant capacity thus saving in total construction cost.

**Key words:** composite structure; stress-transfer; seismic performance; energy absorption

The rigid connection between concrete piers and steel girders is one of the important applications of composite construction. It is widely accepted that the cost of steel-concrete composite structures is cheaper than that of other conventional structures. Also, the performances of such structures are better than those constructed using only one material. This technique has become increasingly popular in highway bridge systems in countries where severe earthquakes are likely to occur. This is because of its lower construction and maintenance costs compared with those of the other kinds of connections. However, one major set back of steel-concrete rigid connections is its complexity in force transfer mechanism. As a result, many difficulties are encountered in designing composite rigid connections. One effective way to overcome this problem is to conduct experiments and establish design guidelines.

In this study, an experimental work on strength and ductility of a new type of rigid connection between reinforced concrete (RC) pier and steel girder is explained. A static cyclic loading test on scaled down specimens is performed for this purpose. The key distinction between this new connection and conven-

tional rigid connections is that the former uses a set of steel plates together with reinforcement bars to transfer moment and shear forces while the latter uses only reinforcement bars for that purpose. The strength of the connection that uses only reinforcement bars depends on the length and a number of extended bars into the connection<sup>[1-4]</sup>. In the new method, steel bars are connected to two moment transferring steel plates via a set of steel ribs welded to these two plates. Another set of plates is provided for shear transferring between the girder and the pier. The plates connected to girder and pier are bolted together to form a rigid connection.

## 1 Specimen

Usually, composite rigid connections consist of steel girders as superstructure and reinforced concrete piers as sub-structure. It is normal practice to design rigid connections at deck-pier connections where the highest sectional forces are likely to occur in continuous bridges systems<sup>[4]</sup>. The rigid connection proposed in this study mainly differs from other conventional rigid connections in that it is to be constructed directly on the RC column. The moment comes from superstructure transfers to RC pier through two steel plates attached to the pier. Another two steel plates are partly inserted to the RC pier for the purpose of shear transferring. These steel plates and RC pier

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act as a single unit. The connection is made by means of high strength steel bolts. This is not the case in conventional composite rigid connections. Also, the proposed new connection can be made in a shorter time since concrete pouring is not needed at the stage of connection being made.

The plate arrangements of the connection are shown in Fig.1. First, two steel plates, as labeled by PL1, as shown in Fig.1, are attached to the pier at the two opposing faces by means of two rows of steel ribs that are welded to each plate. These ribs have holes so that reinforcement bars in the concrete pier can be positioned through these holes, and be anchored. This is the most important mechanism for transferring tension force from the reinforcement bars to the moment transferring plates. Another two steel plates are partly inserted in the pier in order to act as shear transferring plates (see Fig.1). These plates, as labeled by PL2, are welded to the moment resisting plates at their edges. Two types of plates, as labeled by PL3 and PL4, are attached to the girder so that the connection between the girder and pier can be made. A set of additional ribs, as labeled by PL5, is welded to the plate PL3 in order to stiffen them. Holes are provided in all four moment and shear resisting plates so that the connection between pier and girder can be made using high strength bolts.

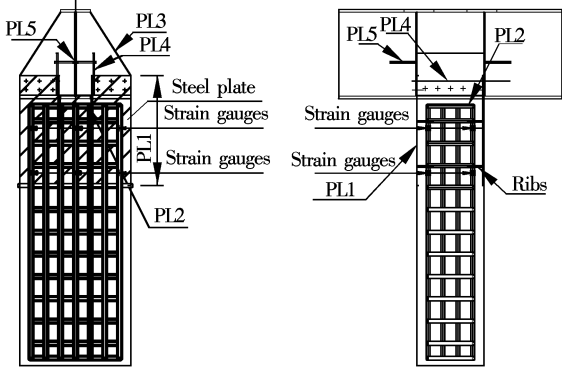


Fig. 1 Details of the connection

2 Experimental Procedure

The test is carried out by applying horizontal cyclic loads at the free end of the RC column, as shown in Fig. 2. An actuator having a capacity of 2000 kN mounted on a rigid steel frame is used for this purpose. The steel girder is firmly bolted to the rigid base. Lateral displacements at several locations along the column are measured using displacement transducers. Strain gauges are placed on the moment transferring plates, shear plates and

reinforcement bars as shown in Fig.2.

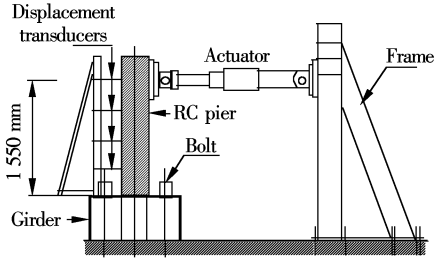


Fig. 2 Test setup

The first lateral load is applied until the steel bars start to yield. This can be confirmed by the strain gauge readings of reinforcement bars. The displacement and load corresponding to this instant are defined as the yield displacement  $\delta_y$  and the yield load  $H_y$ . The cyclic loads are then applied as a multiple of  $\delta_y$  in both positive and negative directions of the column axis (i.e.,  $\pm 1\delta_y$ ,  $\pm 2\delta_y$ ,  $\pm 3\delta_y$ , etc.).

3 Experimental Results

3.1 Strength and ductility of rigid connection

The load is applied until the concrete is crushed. At the stage of the onset of steel bar yielding, the yield load  $H_y$  of 222 kN and yield displacement  $\delta_y$  of 12.8 mm are observed. The concrete crushing occurred at the displacement level of around  $7\delta_y$ . The observed lateral-load versus lateral-displacement hysteretic relation ( $H$ - $\delta$  relation) is shown in Fig.3. The envelope curve of the hysteretic curves is shown in Fig.4. It was observed that the maximum load is about  $1.23 H_y$  ( $= 274$  kN) and the corresponding displacement is about  $1.8\delta_y$  ( $= 23.4$  mm). The ultimate displacement  $\delta_u$  is about  $7\delta_y$ . At this stage,

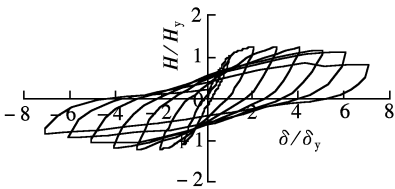


Fig. 3 Lateral load vs. lateral displacement hysteretic relation

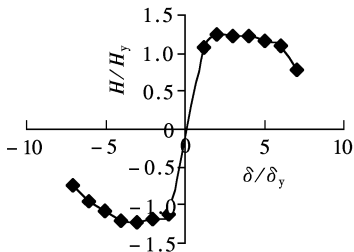


Fig. 4 Lateral load vs. lateral displacement relation

the load decreases by 68% of the maximum load at 186 kN. It is clear that this connection performs well in terms of ductility. The energy absorption of the connection at each cycle is shown in Fig.5. It is seen here that the energy absorption capacity increases up to  $6\delta_y$ . This confirms that the connection has good earthquake resisting performance in terms of energy absorption capacity. This is because the superstructure and substructure act as a unit, hence the combined performance of earthquake resisting characteristics of concrete and steel become much better.

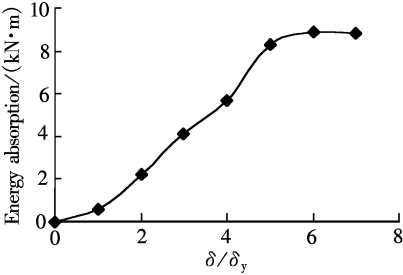


Fig.5 Energy absorption capacity

Currently, no specific design rule is available to design this kind of rigid connection. Experimental results obtained in this study may afford the basis for design rules. Meanwhile, the method used in Japan Highway Design Specification<sup>[5]</sup> specifies the following two equations to calculate  $\delta_y$  and  $\delta_u$  of RC columns:

$$\delta_y = \frac{M_u}{M_{y0}} \delta_{y0} \tag{1}$$

$$\delta_u = \delta_y + (\phi_u - \phi_y)L_p \left( h - \frac{L_p}{2} \right) \tag{2}$$

where  $\delta_{y0}$  is the first yield displacement (m);  $M_{y0}$  is the first yield moment (kN · m);  $M_u$  is the ultimate moment (kN · m);  $L_p$  is the length of plastic hinge (m);  $h$  is the length between the base and the loading point (m);  $\phi_u$  is the curvature at yield stage (1/m), and  $\phi_y$  is the curvature at ultimate stage (1/m). The ductility parameter  $\mu$  is defined as the ratio of  $\delta_u$  to  $\delta_y$ <sup>[3]</sup>. Then, parameters  $H_y$ ,  $\delta_y$ ,  $H_u$ ,  $\delta_u$ , and  $\mu$  of the column in the specimens are calculated according to the above specifications and then are compared with those of the test. The results are presented in Tab.1.

As shown in Tab.1, the ratio of experimental and calculated yield load and yield displacement are respectively 1.24 and 1.14. In many of the previous studies, the yield displacement ratio was between 1.5 and 2.0<sup>[3]</sup>. In the present study, the deformation is restricted due to the presence of steel plates, so the experimental  $\delta_y$  is smaller than those of the other kinds

of connections. In contrast, the  $E/C$  ratio of the ultimate load  $H_u$ , of this study is around 1.40 which is considerably larger than those reported in other studies<sup>[3]</sup> where the ratio was found to be very close to unity. The higher ultimate load observed in this study is attributed to the moment and shear restraint provided by the plates. Similarly, the  $E/C$  ratio of the ultimate displacement  $\delta_u$ , is also quite high ( = 4.22).

Tab.1 Comparison of calculated and experimental results

| Item                   | $H_y/\text{kN}$ | $y/\text{m}$ | $H_u/\text{kN}$ | $u/\text{m}$ | $\mu$ |
|------------------------|-----------------|--------------|-----------------|--------------|-------|
| Calculated value $C$   | 179             | 11.2         | 196             | 21.5         | 1.9   |
| Experimental value $E$ | 222             | 12.8         | 274             | 90.8         | 8.1   |
| $E/C$                  | 1.24            | 1.14         | 1.40            | 4.22         | 4.26  |

The ductility parameter  $\mu$  is calculated from  $\delta_y$  and  $\delta_u$  which are determined from Eqs. (1) and (2). Comparison of test and computed ductility values shows that observed ductility is considerably higher than that of the analytical value. This means that if adopted, the procedure specified in the Japan Highway Design Specification significantly underestimates the available ductility of this new type of connection.

3.2 Force transferring from reinforcement bars to moment transferring plates

As previously explained, the bending moment transferring from the reinforcement bars in the pier to the moment transferring steel plates occurs in terms of coupling forces. The proportions of force taken by the reinforcement bars and steel plates can be estimated by using the strain measurements. The exact locations of the strain gauges on the bars are shown in Fig.6. Here, two sets of strain gauges are placed on the bars in such a way that they are 200 mm and 500 mm below the top level of the pier, respectively (see gauges No.1 and No.2 in Fig.6). The horizontal force and

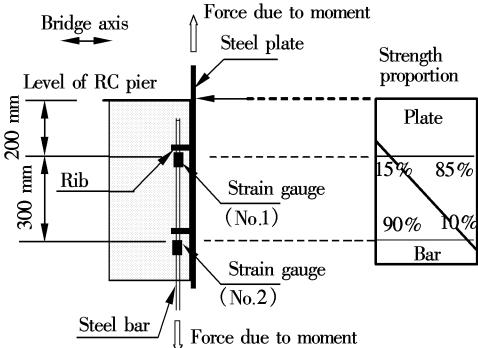


Fig.6 Reinforcement bars in RC pier and proportions of forces in bars and plates

corresponding strain in the strain gauge No.2 is shown in Fig.7. As shown in the figure, the strain in the bars increases with increasing horizontal force. The strain at yield displacement  $\delta_y$  at the first cycle is  $1\,150\,\mu$  and at the fifth cycle about  $1\,920\,\mu$ . These strains are useful in determining the force taken by the bars and the plates.

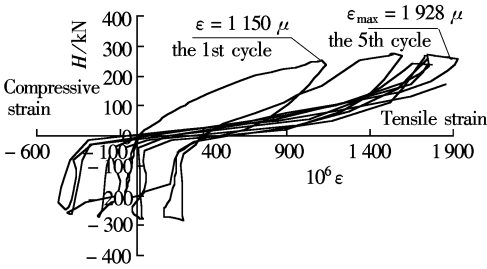


Fig.7 Horizontal force vs. strain in reinforcement bars (at gauge No.2)

If steel at the gauge No.1 gets yielded, then the strain at gauge No.2 should be equal to  $1\,306\,\mu$ . But, the observed strain is about  $1\,150\,\mu$  which is about 90% of the calculated strain. This implies that 10% of force is taken by the steel plates at 500 mm below the top level of the pier. The strain at the level of strain gauge No.1 is expected to be equal to its yield strain if no plates are provided. But, the actual strain at this location is  $287\,\mu$  which is about 15% of its yield strain. So, 85% of forces have been transferred to steel plates. These proportions are graphically illustrated in Fig.6.

3.3 Force transferring through shear plates

The proportion of horizontal force taken by the shear plates can be estimated using shear strain measurements obtained from rosette strain gauges placed in the plates PL4 as shown in Fig.1. Then, the shear strain,  $\gamma_{xy}$ , is computed by

γ<sub>xy</sub> = - (ε<sub>x</sub> + ε<sub>y</sub>) + 2ε<sub>45</sub> (3)

where ε<sub>x</sub> and ε<sub>y</sub> are the strains in x and y direction, and ε<sub>45</sub> is the strain in direction 45° inclined to x-direction. Fig.8 shows the observed horizontal force and computed shear strain relationships. The force taken by the shear plates (Q) can then be easily calculated using material and geometric properties of steel plate as follows:

Q = (Gγ)A (4)

where G is the shear modulus and A is the cross sectional area of the plate. The observed and computed values of shear forces and their ratios are presented in Tab.2. Here, the values corresponds to two stages (i.e., bars at yield stress and bars at ultimate state)

are presented. As seen in the figure, shear plates take about 80% of the horizontal force.

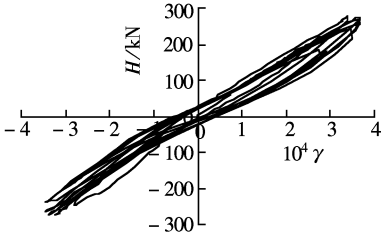


Fig.8 Horizontal force vs. shear strain

Tab.2 Computed and experimental shear forces

| Item                   | Horizontal force H/kN | Shear strain in plate γ  | Shear force Q/kN | Shear force ratio Q/H |
|------------------------|-----------------------|--------------------------|------------------|-----------------------|
| Bars at yield          | 222.0                 | 2.828 × 10 <sup>-4</sup> | 168.0            | 0.76                  |
| Bars at ultimate state | 274.0                 | 3.665 × 10 <sup>-4</sup> | 217.0            | 0.79                  |

4 Conclusions

Based on the experimental observations and comparisons of actual and computed strength and ductility parameters, the following important findings can be made from this study:

- 1) The moment from superstructure can be effectively transferred to substructure through set of moment transferring plates with holed stiffeners in terms of coupling forces.
- 2) The plates provided for shear transferring can withstand up to around 80% of total horizontal force.
- 3) The energy absorption capacity of this connection is good even at large displacement levels such as 7δ<sub>y</sub>.
- 4) The locations where stress concentrations occur do not experience any buckling. Thus, repair work is convenient.

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# 钢梁与钢筋混凝土桥墩的新型刚节点 联结形式及其静载反复试验

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**摘 要** 提出了一种新型的桥梁钢梁与钢筋混凝土桥墩的刚节点联结形式. 刚节点内部构件间的连接全部采用高强度螺栓连接方式且施工方便, 能够较理想地将上部结构弯矩传递到下部结构, 剪力传递钢板承受了大部分的水平剪力. 通过静荷载反复试验获得了有关这种新型结构性能方面的有效数据, 试验结果表明该结构内部应力传递路径明确, 并具有良好的抗震性能. 有关结构的强度和塑性变形的计算及试验结果均表明, 这种结构即使在发生很大变形的情况下, 也能保持良好的能量吸收功能且没有出现因应力集中而发生的局部屈服现象. 由于避免使用了价格昂贵的支座, 采用这种新型组合结构形式不但具有良好的抗震性能, 同时可以降低桥梁的建设成本.

**关键词** 组合结构; 应力传递; 抗震性能; 能量吸收

**中图分类号** TU398