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Verifying Computer Models of Bridge Foundations

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SUMMARY
The results produced by Computer Models are governed by the boundary conditions introduced by the Engineer. It is common practice to define springs that describe the behavior of the foundation. In most of the cases it is assumed that the foundation is rigidly connected to the ground. Actually soil structure interaction practically occurs in every project and may change the results considerably. To verify a computer model monitoring of the vibrational behavior of the structure might be applied. By the application of System identification technologies the real behavior of the structure is determined and information on the acting boundary conditions is achieved. This state of the art technology has been developed recently. One example of application is provided in this paper.

1 KAO PING HSI BRIDGE

The Southern Second Freeway, which extends from Kaoshiung to Ping Tung crosses the Kao Ping River along it's route. A major bridge project is required to span this wide and, under Typhoon conditions, violent river. The total length of the required bridge is 2600 meters with a major span crossing the river bed. The proposed **Kao Ping Hsi Bridge** was designed to fulfill the traffic requirements in an environmentally sensitive way.

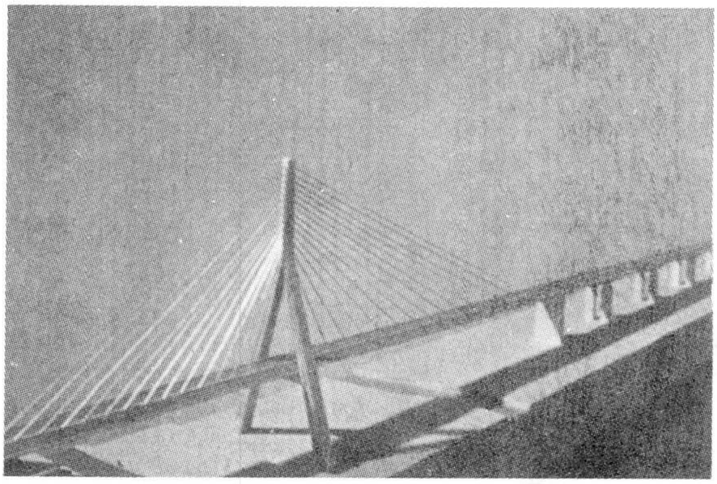

Fig.1 Kao Ping Hsi Bridge

The proposed structure is a cable stayed bridge with an unequal span arrangement. The best location for the pylon was determined with respect to geometry and the relation of the structure to the natural environment. This produces a final side span length of 186 meters. For a perfect geometrical balance the main span was chosen to be 330 m. This is also appropriate relative to the height of the structure, which is an average of 45 meters above the ground.

The height of the pylon was fixed at 180 meters. For aesthetic and balancing reasons, it was decided not to use anchor piers in the side spans. This makes the structure unique and one of the largest asymmetrical cable stayed bridge in the world.

To help balance the span, it was decided to use a concrete deck for the side span and the steel deck for the main span. Fourteen pairs of cable support the structure in a semi-fan arrangement. The cables are anchored on both ends and the distance between cables was chosen to maximize utilization of the deck's bending capacity and optimum stabilization of the pylon.

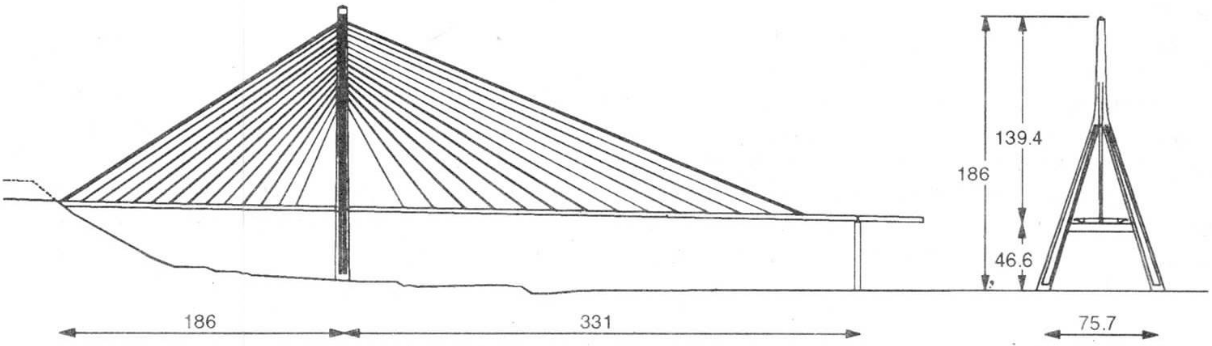

Fig.2 Bridge Elevation

2 PYLON FOUNDATION

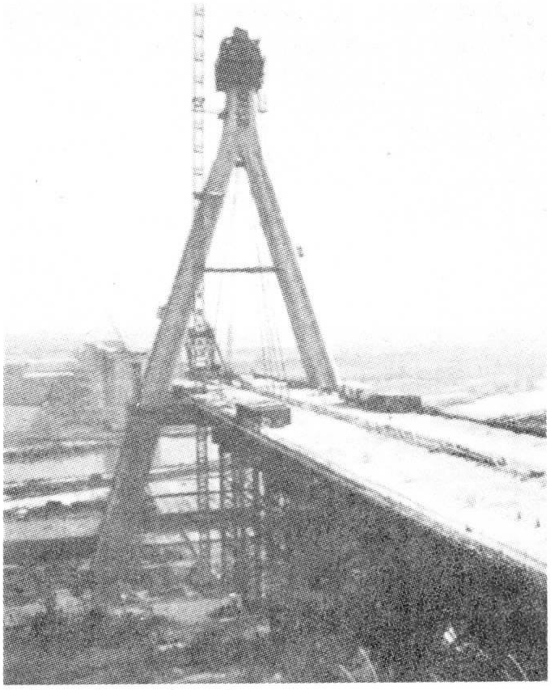

Fig.3 Kao Ping Hsi Pylon

In the course of the preliminary design, two variations were considered for the pylon foundation; one foundation with slurry walls and the other with piles of ∅ 1.50 m. Under the observance of the dispersing forces, as well as the scouring, an efficient comparison of both variations was executed, whereby it was shown that a slurry wall foundation is the more favorable solution from the economic point of view. Additionally, this system results in a substantially greater security of the foundation elements.

The basic principle of the foundation in the presence of the soil types as determined by the borehole samples, is that:
The soil within the foundation elements is enclosed through the slurry wall, the normal forces are dispersed correspondingly during lateral friction, and, through the corresponding stiffening of the slurry wall elements, as also through structural design of the individual elements, an optimal dispersal of the horizontal forces in a normal loading condition, as well as in an earthquake loading condition, is secured.

There are two proposals: one for slurry walls with a diameter of 0.80 m, and the second for slurry walls with a diameter of 1.20 m.

2.1 Basic Concept

Normal forces and both shear forces and bending moments each acting in the longitudinal as well as the lateral direction of the bridge, occur at the underside of the foundation slab.

2.2 Normal Forces

The dispersal of the normal forces results during lateral friction, whereby the lateral friction begins at a level of +11.0 m. This measure takes under consideration complete scouring in the event of a catastrophe. The lateral friction itself is applied internally and externally, whereby the spacing of the individual elements amounts to a minimum of three times the width of the element's cross section. The value of the lateral friction amounts to 65 kN/m^2. This value follows from accumulated values based on borehole samples and results of the Standard Penetration Tests.

Fig.4 Foundation

2.3 Shear Forces – Longitudinal Direction

Shear forces in the longitudinal direction of the bridge are dispersed over an elastic bedding at the base. For the normal loading condition, a bedding according to the figure below is considered. Additionally it is important that above all a corresponding safety factor is given for the earthquake loading condition in relation to the possible passive soil pressure. For the earthquake loading condition itself, a full scouring must not be considered alone. Moreover, the possible passive soil pressure at total scouring plus earthquake loading was also analyzed, thereby showing that nevertheless the second degree safety factor is still present.

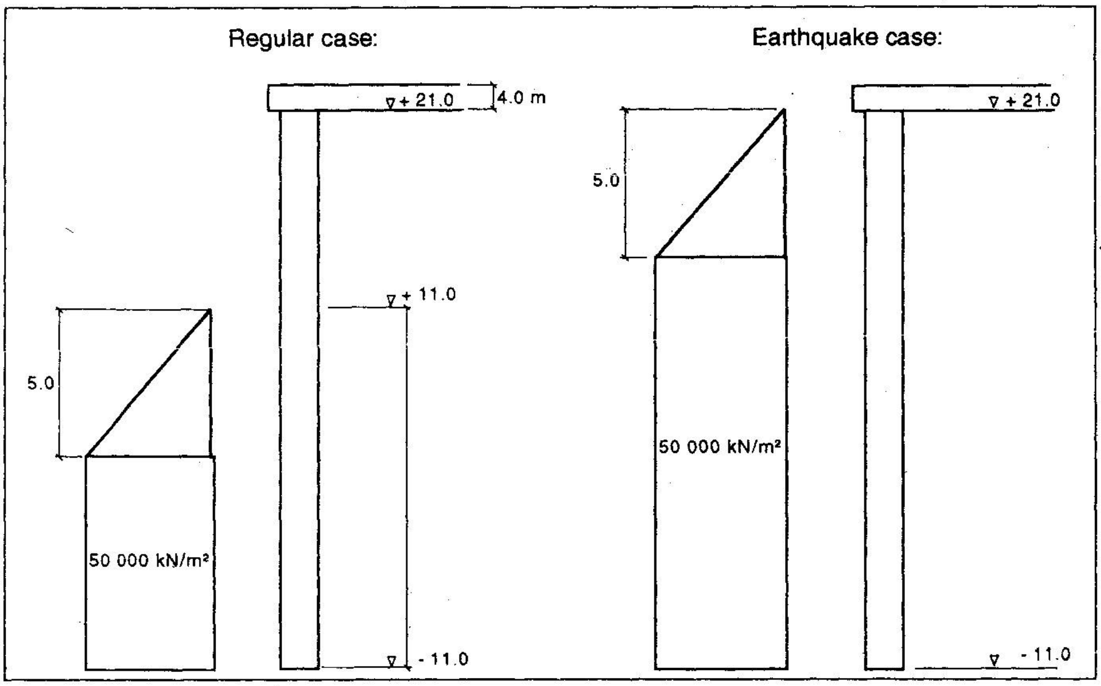

Fig.5 Horizontal bedding – pylon

3 VIBRATION CHARACTERISTIC METHOD

Each structure has its typical dynamic behaviour which may be addressed as „Vibrational Signature". Changes in a structure such as all kinds of damages leading to decrease of load-carring capacity have effects on the dynamic response. This suggests the use of the dynamic response characteristic for evaluation of structural integrity. Monitoring of measurements of the dynamic response of structures makes it possible to get very fast knowledge of their actual condition.

The process of elaborate instrumentation, measurement and analysis of dynamic response data supplemented by immediate or long-term maintenance and rehabilitation programs can be considered as „health" monitoring and diagnostic operations.

The development of a - for the user simple and cheap - method for permanent control of the state of preservation based on the dynamic behaviour of structures makes access to a couple of specialized know-how necessary.

Before the installation of measuring instruments analytic investigations have to be done. Therefore the knowledge of the dynamics of structures is used and adapted to the particular problems. Several dynamic calculations of large complex civil structures especially bridges have been done by many research institutes and are reported in technical journals and on workshops.

The outcome of these calculations is the theoretic dynamic behaviour of the analyzed structures which makes the development of special measuring programs possible. Among dynamic measurement units some measuring instruments for registering environmental influences such as for example temperature will be necessary.

4 INSTRUMENTATION AND MEASUREMENT

Already during design it was decided to instrument the bridge at construction and after completion. This includes a number of strain gages for permanent installation and a number of accelerometers for temporary use and final installation in the structure. The main purpose was quality control and system identification for the calibration of the Computer models.

For the verification of the soil springs in the Computer model the Eigenfrequencies and Eigenmodes of the free standing pylon were determined. The first measurement was taken when an elevation of +45 meters has been reached. At this time the high rigidity of the structure required amplification of the ambient signals for correct computation. Nevertheless the required results could be obtained at the correct spring value could be determined.

Location	Theoretical frequency	Measured frequency	Difference
At level +45 m	5.66 Hz	5.21 Hz	-8%
At level +120 m	1.22 Hz	1.09 Hz	-11%

These values are given for the most effective natural frequency measured. The spring values determined from this measurement has been tried with the Computer model and the real displacement of the pylon under the load of the first cable has been measured. The results matched within 10% accuracy, which is very good.

5 CONCLUSION

System identification technology can help to calibrate Computer models accurately. Particular in case of very flexible structures this tool becomes helpful for the designer to adjust his calculation and detect deviations from the normal conditions. Further development is put into more elaboration of the method by the European Union and tools applicable for everyone will be available soon. In some cases it will lead to cheaper solutions in the foundations because the reserves, not activated until now and covered by high safety factors, might be touched in future.

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A Study on the Techniques of Spiral Pressure and Cementation for Underwater Concrete Piles

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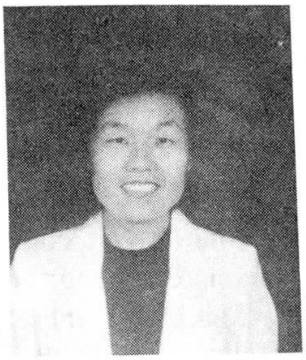

Tong Liping, born in 1960, got her civil engineering degree of PhD at Southeast University of China. She has been working on the teaching and research of civil engineering for many years.

SUMMARY

The technique of pressure and cementation for underwater spiral concrete piles discussed in this paper is a new research on pile foundation made by the authors in recent years. It adopts an advanced positioning method for spiral hollow piles drilling below the solid soil layer by means of compactness and rotation, and then in the process of depressive, static and upward rotations, concrete is being poured down until a compacted pile body has been formed.

There are many advantages for this pile technique. Firstly, this technique can hardly be influenced by groundwater impulsive force and, consequently, the formation of piles is greatly sped up. Next, the system of force transmission is practical and reliable. Since the spiral piles are screwed into the ground, it has little disturbance on the layers and increases the frictional resistance of soil around the piles, and thus the pile capacity is greatly enlarged. In the third place, the static rotation below the hard layer broadens the area of a pile point and makes the layer so closely adhesive to the pile that the layer resistance of piles become strong. Finally, the application of pressure and cementation technique can increase the adhesion of soil around the piles and reduce the difficulty of construction.

In conclusion, underwater spiral pile, by using the technique of pressure and cementation, have the advantages of advanced production process, large load capacity, fast speed of forming piles and some other effective comprehensive benefits. The study on the computational theories and pile-forming processes is an important task. If widely used, this technique is bound to bring good social and economic benefits to civil engineering.

1 INTRODUCTION

With the development of civil engineering, pile foundation has been widely used in high-rises, bridges, harbours or ports, ocean platforms and nuclear power plants, etc. New modes of piles, new techniques and new pile materials are increasingly widely being used in pile engineering, greatly improving design and computation theories, construction methods, full-scale tests and detection solutions.

On the other hand, the cost of pile foundation takes up over 25% of the total cost in a building construction. From structural point, pile foundation bears all the loads of a building including self-weight, dead and live loads. Therefore, any damage to pile foundation will bring about unexpectable danger and reduce the reliability of a structure to a great extent. Thus, it is an urgent task to develop a mode of pile foundation with high quality, low cost and fast speed of construction.

Concrete filling piles vary with the condition of engineering geography according to their design bearing capacity, length and diameter. Since the needed equipment of construction is simple, the method is flexible and the cost is low, these piles have been coming into wide use in all the fields of civil engineering. However, since the quality system of these piles is hard to keep, there are some quality accidents happening such as diameter compactness, pile break-down, partial mud mixture, concrete segregation from pile bodies, top porosity, etc.[1]

The technique of pressure and cementation for underwater spiral concrete piles discussed in this paper is a new process of forming piles researched by the authors in recent years. The construction process includes: positioning→ sinking rotation→ static rotation→ rising rotation→ pouring rotation→ finished piles. It adopts an advanced poisoning method to screw the hollow pipes into soil layers by means of pressure and rotation. After sinking rotation, use static rotation to enlarge the pile point, and meanwhile, pour concrete with the hob starting drilling. Under the condition of pressure and cementation, a contracted pile body is formed and the quality system is fully guarantied. In addition, this process can be hardly influenced by groundwater or water impressive force, and consequently, the formation of piles is greatly sped up. This paper uses the method of mathematical statistics to analyze the reliability of each chance variable influencing the bearing capacity of pile foundation. As a result, the system of force transmission of spiral piles is reasonable and reliable. The design theory and construction technique have been used in practical engineering construction and the result obtained from the full-scale tests tallies with that of theoretical analysis.

2 ENGINEERING EXAMPLES AND TEST RESULTS

2.1 Engineering Survey

Engineering name: Building of China Henan Test Center of Forest Reserves, length 48.6m,width 12.9m, land-holding area 627m^2, 7-story frame structure. The topography of the site is smooth with simple geomorphy, belonging to Yellow River flooding and leveed plain. According to a report of engineering geology, there are nine soil layers in this area, of which the layers from 1 to 8 is plastic and saturated , with high groundwater elevation and complex geological condition.

The design requirement for pile foundation is underwater concrete piles using the technique of spiral pressure and cementation with C15 of concrete intensity scale, 11.0m of effective design length, 400mm of diameter and 200kPa of standard design value of compound bearing capacity of pile foundation.

2.2 Analysis of Test Result[2]

Test content: Dead load experiment—3 groups for bearing capacity of single piles and 2 for that of foundation slad ; 84 piles for testing the quality wholeness of pile bodies.

Test result: choose 3 groups of 9 piles for dead load experiment of single piles . As a result, there are 7 piles working well without imperfection and 2 working nearly well. The maximum applying load value is 400kN without any limit load. The basic bearing capacity of each pile is all more than the design value, which tallies with the analytic result of serial distribution in mathematical statistics. Through the test on 4 points, the standard value of bearing capacity of compound foundation is 228kPa. In the pile body test of quality wholeness, 94% of the 84 tested piles belong to class A (high quality without any imperfection), 4.8% belong to class B (close to high quality but having slight imperfection) and 1.2% belong to class C (low quality with much imperfection). Therefore, the design method and construction technique for underwater concrete piles are able to fully meet the need of quality system of pile bodies.

3 ANALYSIS FOR THE RELIABILITY OF PILE BEARING CAPACITY

3.1 Properties of Probability Statistics of Any Factor Affecting the Bearing Capacity of Pile Foundation

3.1.1 Probability Model of Pile Load on the Pile Top [3]

Dead load usually obeys normal distribution and its probability distribution function is

$$F(x) = \frac{1}{\sigma_G \sqrt{2\pi}} \int_{-\infty}^{\infty} \exp[\frac{(u-\mu_G)^2}{2\sigma_G{}^2}]du \tag{1}$$

μ_G , σ_G are respectively dead load average and standard deviation.

The probability distribution of variable load can be described by limit value I and its probability density function and distribution function are

$$f(x) = \alpha \exp\{-\exp[-\alpha(x-u)]\{\exp[-\alpha(x-u)]\}\} \tag{2}$$

$$F(x) = \exp\{-\exp[-\alpha(x-u)]\} \tag{3}$$

where the parameters α and u are respectively

$$\alpha = \frac{\pi}{\sqrt{6}\sigma} \qquad u = \mu - \frac{0.5772}{\alpha} \tag{4}$$

μ , σ are respectively load average and standard deviation.

Wind load also obeys the limit value I distribution. As for anti-seismic structure, we also need to consider seismic load.

3.1.2 Intensity of Pile Material and Probability Model of Sectional Area

Choosing proper concrete intensity scale is quite important to give full play to the bearing capacity of force-resistance soil layers of piles and to increase the vertical and horizontal bearing capacity of a pile itself. The bearing capacity R of a single pile can be calculated as follows,

$$R = \varphi f_c A_p \tag{5}$$

Where f_c, A_p are respectively the pressure-resistance strength of concrete and the sectional area of a pile body; φ is vertical curved coefficient.

3.1.3 Statistical Parameters of Limit Bearing Capacity

When a load applies to a pile top, the upper part can produce a downward displacement against soil layers caused by compression, and meanwhile the surface of a pile receives frictional resistance from soil layers. The compression and displacement of piles increase as loads do and the frictional resistance also increases in lower part so that the bottom layer will produce resistance caused by compression. When the resistances of a pile body and bottom both reach a limit value, the load capacity against the pile is called limit bearing capacity. The main factors influencing the limit bearing capacity of pile foundation include pile diameter, length, thickness of each soil layer, limit frictional resistance and the variability of limit bearing capacity at the bottom of soil layers.

The standard value R_k of the bearing capacity for a single underwater pile can be calculated as follows,

$$R_k = \pi d \sum_{i=1}^{n} l_i \tau_i + Aq \tag{6}$$

d, A are respectively the diameter of piles and the bearing capacity of pile bottom; l_i, τ_i are the layer thickness around a pile and its limit frictional resistance.

3.2 Analysis of Reliability for Piles

There are two modes of damage for piles under the load effect of pile foundation. One is that pile material cannot bear the upper load so that the pile body is damaged, and the other is that, although the pile strength is large enough, the bearing capacity of the soil around and at the bottom of the pile is not large enough. This paper discusses the reliability of each failure mode and then calculates the reliability of pile foundation.

3.2.1 Reliability Modes of Load Effect S and Bearing Capacity R

When the bearing capacity R of pile foundation is smaller than the load effect S, the pile will be damaged and the reliability of pile foundation is

$$P_s = P(R - S > 0) \quad (7)$$

3.2.2 Reliability Mode of Single Piles

Since pile foundation can maintain its normal function only under the conditions of $R_k > S$ and $R > S$, and meanwhile both $R_k > S$ and $R > S$ are independent events from each other, single pile reliability can be expressed as follows,

$$P = P(R - S > 0, R_k - S > 0) = P_s(R - S > 0) \cdot P_k(R_k - S > 0) \quad (8)$$

3.3 Statistic Inference of Probability Mode with Limit Bearing Capacity

In addition to the influence of pile length, diameter and concrete intensity scale, the bearing capacity of underwater concrete piles with spiral pressure is mainly influenced by some uncertain factors like raw material, construction technique, site environment and mechanical properties of soil layers, etc. It is very difficult to describe such a large number of factors by present mechanically analytic modes, of which the single pile limit bearing capacity can be regarded a comprehensive reflection. Therefore, this paper uses the limit bearing capacity of single piles to evaluate the quality of piles.

4 CONCLUSION

As shown from the above theoretical analysis and the analysis of real engineering test, the technique of spiral pressure and cementation for underwater concrete piles is a process of pile formation with high speed, high bearing capacity and compound effective benefits. Compared with traditional filling piles, class A piles increases by 12%. Therefore, we can obtain the following conclusions:

(1) Since the spiral piles are screwed into the ground, they not only have little disturbance on the layers, but increase the frictional resistance of soil around the piles as well so that the pile bearing capacity is greatly increased. On the other hand, the static rotation below the hard soil layers broaden the area of a pile point and make the layer so closely adhesive to the piles that the layer resistance of piles becomes strong.

(2) The application of pressure and cementation technique can increase the adhesion of soil around piles and reduce the difficulty of construction.

(3) The process from position to downward rotation to upward rotation makes the concrete pressed into hob untouch ground water or layer mud with a drill rotating continuously, and the concrete pouring down the hob is in constant agitation, which avoids partial mud mixture or segregation of concrete. In addition, since the spiral pressure of drilling pipes makes concrete

poured into the pipes drop fast, there is minimum probability of diameter compactness and pile break-down.

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Rapid construction of railway bridge by soil cement and steel pile method

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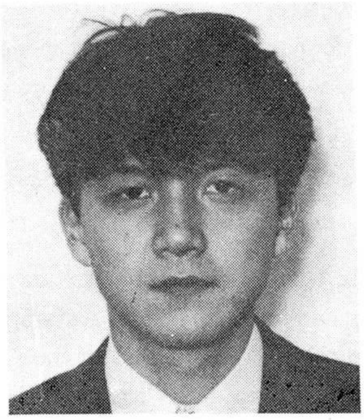


SUMMARY

In The Akita Sinkansen Line that started operation in March '97, the new construction of its elevated viaducts which connects to the elevated bridge of Tohoku Sinkansen Line had to be rushed in half a year in city area.
In the competent place, soft ground(N-value:under10) accumulates under the gravel bed that is supposed to be bearing stratum of spread foundation, and pile foundation was adopted.
The pile foundation could be constructed with little noise and shock, rapidly, and it was necessary to excavate the ground including boulder. So, it was impossible to adopt driving pile, pile installation by inner excavation, and cast-in-place pile. For that, a friction pile by soil cement and steel pile method with rock auger was adopted as a part of viaduct foundation for the first time as railway structure. This method differs from ordinary method in using rock auger method with steel pile casing to crush and excavate underground obstacle.
In real construction, the diameter of excavation was 1000 millimeters, the length of soil cement was 16.5 meters. Before real construction, vertical loading test was enforced to confirm bearing capacity by means of test pile. Through four-thirds the load of design that was loaded, settlement of pile top was 13 millimeters. In real construction, 42 piles were constructed for 20 days with two machines. This progress speed is about twice that of overall casing method.
This paper reports about the selection of foundation type in the competent viaduct area, the outline of this method, the result of the pile construction test and vertical loading test, and construction.

1 INTRODUCTION

In the Akita Shinkansen Line that started operation March '97, the new construction of its elevated viaducts which connects to the elevated bridge of the Tohoku Shinkansen Line had to be rushed in half a year in city area. The viaducts of the Morioka approach are in the north of Morioka St, are by through trains between Tokyo and Akita as they go from the Tohoku Sinkansen Line to the Tazawako local line.

For that, a friction pile by soil cement and steel pile method with rock auger was adopted as a part of viaduct foundation for the first time as railway structure.

This paper reports about the selection of foundation type in the competent viaduct area, the outline of this method, the result of the pile construction test and vertical loading test, and construction.

2 SELECTION OF FOUNDATION TYPE

2.1 The Morioka approach

The extension of the Morioka approach is 1.2km (Fig.1). The extension of the viaducts is 1.0km. The houses stand close together around that. In the area, we had to construct the Akita Shinkansen Line over the Tazawako Local Line. We devised to construct foundations and pillars on both sides of the railroad, connect that two pillars, and erect a beam. But in that method, the construction expenses was high. It was difficult to secure the road for construction, the construction period was long. So, we have suspended train service in the Tazawako Local line for an year, constructed the viaducts for six months.

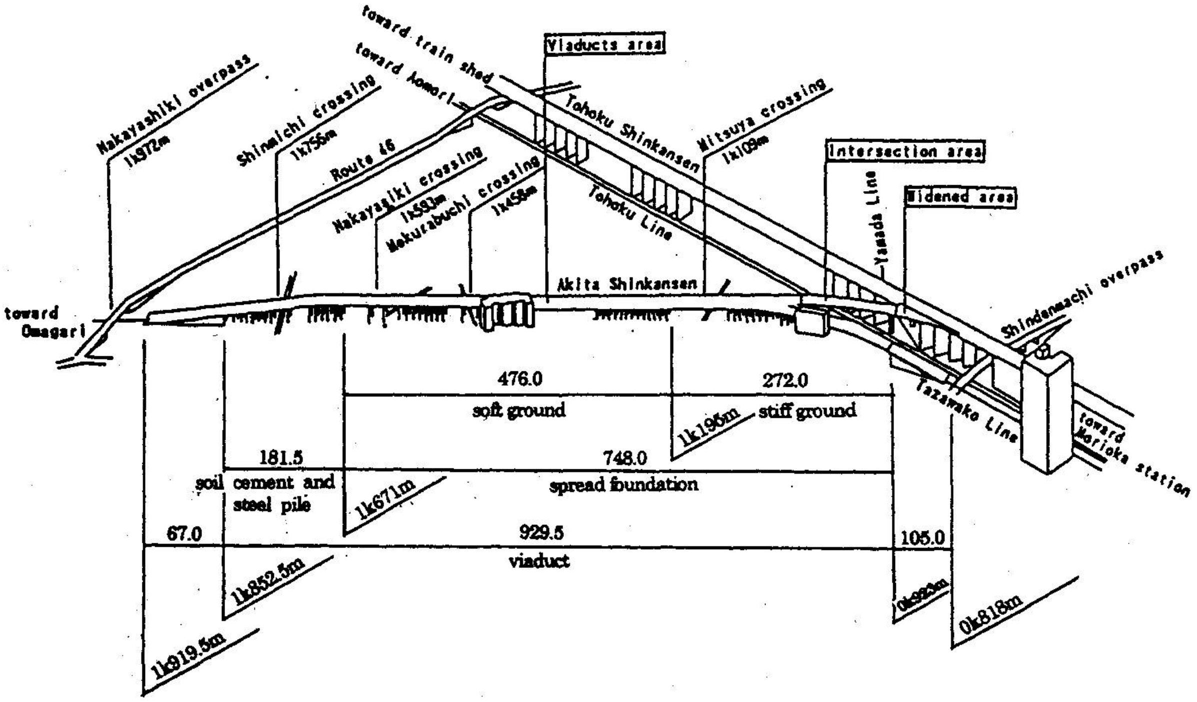


Fig.1 The Morioka approach near Morioka Sta.

2.2 Selection of foundation type

2.2.1 Topographic features and geological condition

In the viaducts area, geological survey and plate loading test were enforced. We planed to construct spread foundation from starting point to 1k671m. But except that area, sandy clay including gravel (N-value : under 10), the thickness was 3 or 6m, accumulated under the gravel bed (N-value : about 20) which we supposed as bearing stratum of spread foundation, so we adopted pile foundation in fear of consolidation settlement (Fig.2).

And, diluvial gravel bed that was bearing stratum was inclined, uneven. Medium alluvial gravel bed included boulder, the maximum size of that was 150mm.

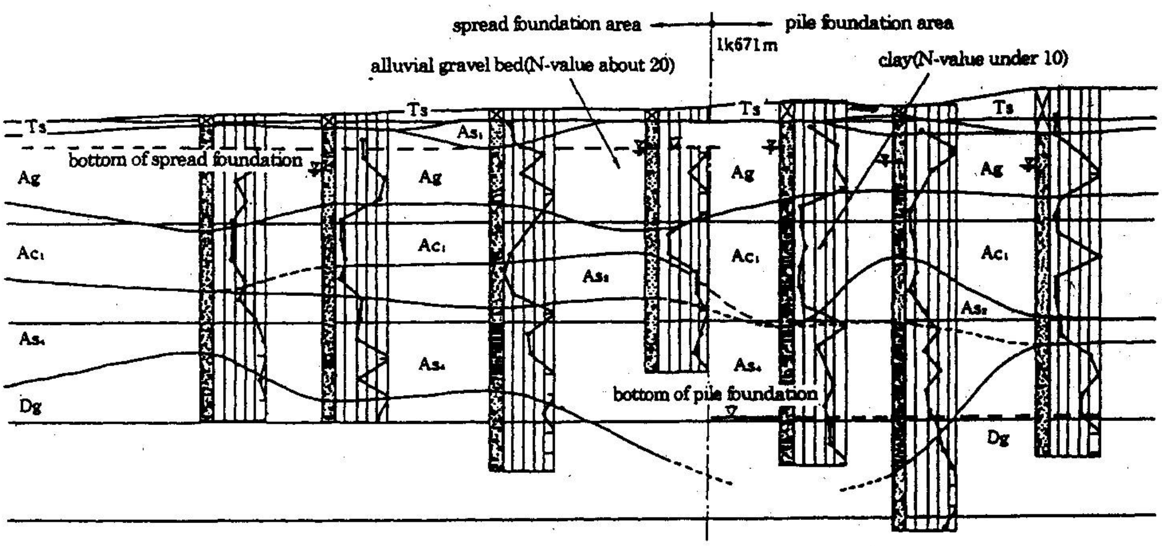


Fig.2 Geological longitudinal profile

2.2.2 Selection of pile method

The viaducts was rigid-framed structure of one pillar and one pile. Pile diameter was 1m, pile length was 15m. Medium stratum included big boulder and gravel. The houses stand close together around there, we had to construct rapidly two piles per a day by one pile driving machine. By those conditions, we selected pile the method.

We evaluated popular pile method as follows.

Driven pile method : Because this method has noise and vibration, we can not use it in city area.

Pile installation by inner excavation : Medium stratum includes big boulder, so it is impossible to construct by this metod. It depends on end bearing capacity of a pile, so it is unsuitable for rapid construction.

Cast-in-place pile method : Overall casing method has noise and shock. In reverse circulation drill method, and earth drill method, working efficiency declines when big boulder and gravel are.

So, we examined to adopt soil cement and steel pile method using steel casing pile and rock auger together, that could construct rapidly, excavate easily without removing boulder, gravel and obstacle in a ground.

3. SOIL CEMENT AND STEEL PILE METHOD WITH ROCK AUGER

3.1 The feature of this method

3.1.1 Construction order (Fig.3)

① Excavation to pile edge by rock auger with steel casing pipe. Grouting cement milk, and first agitating.

② Second agitating, taking down it to pile edge again after pulling up casing to ground surface with turning it. And pulling auger with grouting cement milk.

③ Positioning of steel pile with rib.

④ Completion.

3.1.2 Feature

This method differs from ordinary method in the following.

Excavation machine : Rock auger method with steel pile casing to crush and excavate underground obstacle (cobblestone, a cloud of concrete) is used.

Mouth pipe : For rapid construct, this method excavates, grouts and agitates from surface directly without using mouth pipe.

Foot protection : For rapid construct, foot protection of end of pile isn't performed. This pile is skin bearing pile.

Fig.4 shows the general drawing of this soil cement and steel pile.

Because this method was adopted for the first time for railway structure, construction test and vertical loading test were done. That purpose was as follows.

Construction test : This test confirmed the result of the pile in that ground and the construction condition, and progress speed.

Vertical loading test : This test confirmed the bearing capacity, skin friction in particular, of the pile constructed by this method.

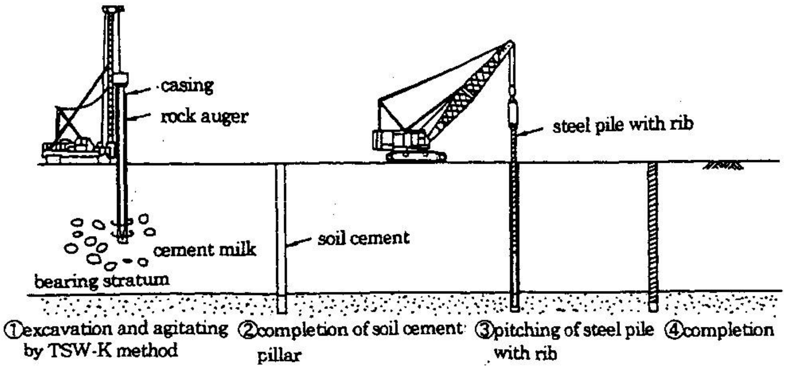


Fig.3 Construction order

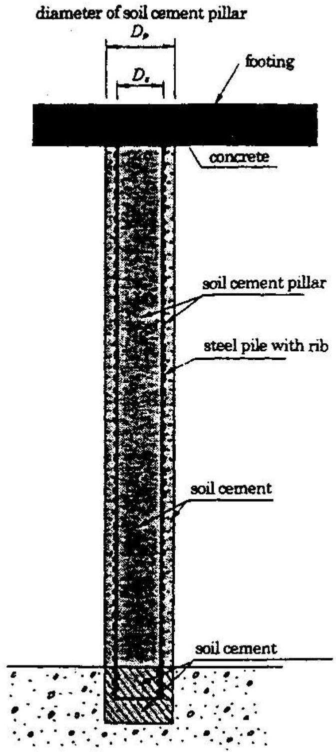


Fig.4 The general drawing

3.2 Construction test

The diameter of the test pile was 1000mm, the length of soil cement was16.5m. We adopted the steel pile, the diameter was 800mm, the thickness was 9mm, without foot protection. That had rib inside in the section of 2m from pile edge. We set up strain gauges in turning point of geology to measure linear stress when vertical loading test was done.

After vertical loading test, we dug out the pile, and surveyed shape of the pile. We confirmed the fixed pile. Table.1 shows the result of progress speed. It took 180 minutes from excavation to positioning of pile. In real construction, it takes time to move machine and quality control etc, so it is need to take some measures for progress speed, two piles per a day. This progress speed is about two times as fast as popular overall casing method.

Table.1 The result of progress speed

Kinds of working		Time(minutes)	
Excavation	Excavation/grouting	72	115
	Agitating	43	
Pitching of pile	Setting out/set staging	45	65
	Pitching of steel pile	20	
Total		180	

3.3 Vertical loading test

This test was done by multi-cycle form, based on the rule of Japanese Society of Soil Mechanics and Foundation Engineering. Planning ultimate load was 930t, four-thirds the ultimate bearing capacity of cast-in-place pile that was based on the design rule of railway structure. Table.2 shows the calculation result of ultimate vertical loading capacity.

Table.2 The calculation result of ultimate vertical loading capacity

Soil	Depth	Thickness of soil L_i(m)	N-value		Cohesion (tf/m²)	Diameter of soil cement ϕ Circumference U Sectional area A_p	f_i (tf/m²)	L_if_i (tf/m)	q_dA_p (tf)
Ts	0.30	0.3	—		—	ϕ=1.0m U=3.14m A_p=0.5026m² 16.5m	—	—	—
Ag	7.30	7.0	26				13	91	
Ac_1	12.30	5.0	8		7.0		7	35	
As_2 Dg	15.20	2.9	30		—		15	43.5	
Dg	16.00	0.8	12				6	4.8	
Dg	17.40	1.4	37	30				ΣL_if_i=174.3	150.8
Dc	19.20	1.8	23						
Dg	Under24.00		40				Subtotal	$U\Sigma L_if_i$=547.3	
							total	698.1tf	

Table.3 shows mix proportion of cement milk. When loading test was done, the unconfined compressive strength of the set cement (age : 43 days) was 30.7kgf/cm^2, the modulus of elasticity E_{50} was 10,000 kgf/cm^2 on an average. Fig.5 shows *P*-*s*-*t* curve. Fig.6 shows log*P*-log*s* curve. We judged ultimate load from log*P*-log*s* curve. Ultimate load was larger than planning ultimate load, 930t. When we loaded planning ultimate load, settlement of pile top was a little 13mm. This showed that constructed pile had sufficient bearing capacity.

Table.3 Mix proportion of cement milk

Unconfined compressive strength q_u(kgf/cm^2)	Cement C(kg)	Bentonite B(kg)	Water W(kg)	W/(C+B) (%)	Retarder (%)	Quantity of grouting
over10	400	20	749	178	0.6	0.884

Fig.7 shows the distribution of linear stress. When we loaded ultimate load, 10 percent of that, about 90t, was transmitted to the end of pile. We grasped that skin friction worked well, and that skin friction corresponded to the ground strength of stratum. The skin friction calculated from this distribution of linear stress was larger than assumed that in all stratum. In clay stratum that we feared the decline of skin friction by excavation and agitating stir with casing, the skin friction satisfied expectation value.

As mentioned, we realized that we could design the pile constructed by this method as skin friction pile, could apply the estimation formula of cast-in-place pile of the design rule of railway structure to estimate this ultimate fearing capacity.

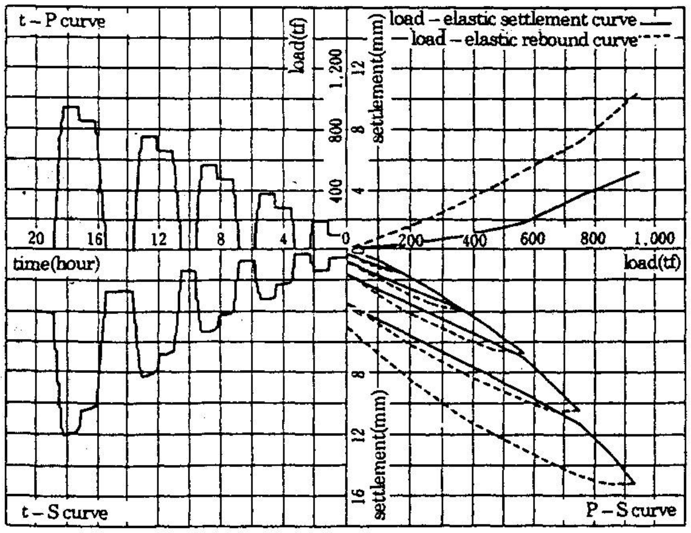


Fig.5 *P*-*s*-*t* curve

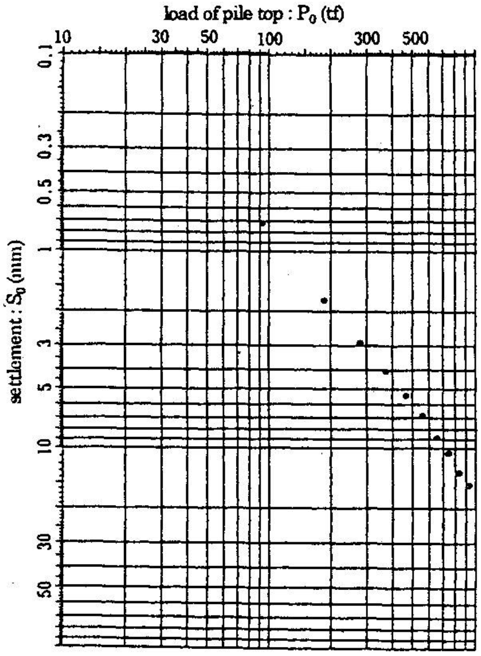


Fig.6 log*P*-log*s* curve

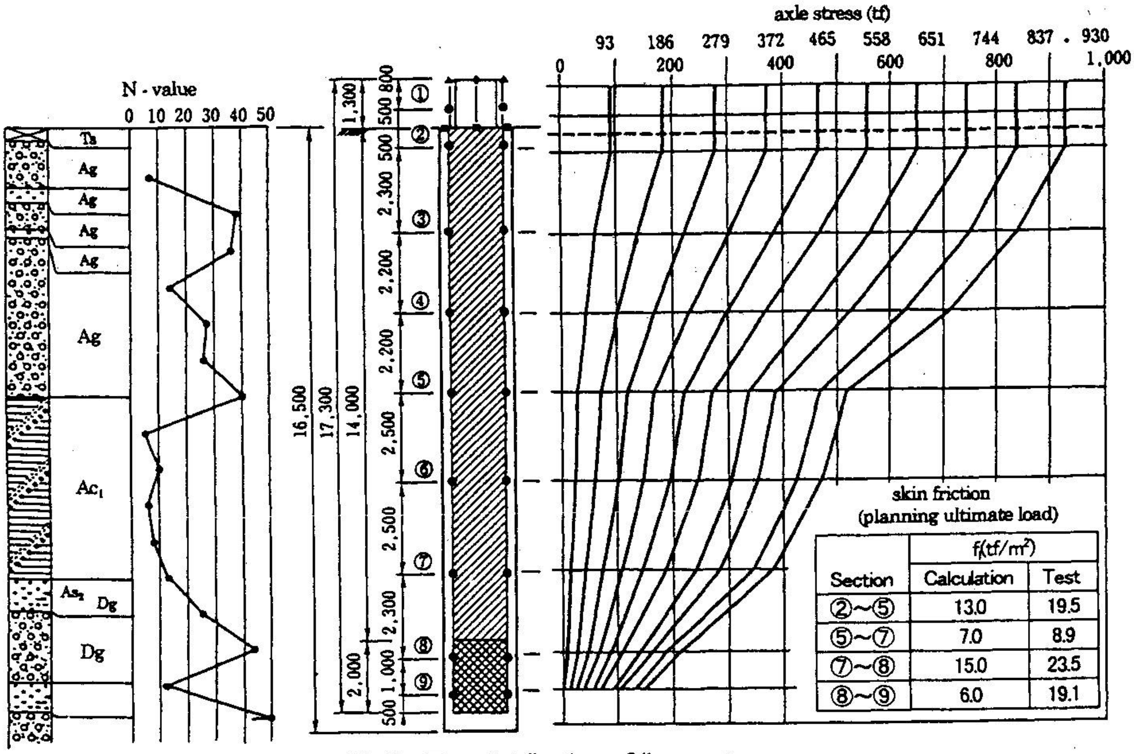
4 REAL CONSTRUCTION

4.1 Cycle time

In real construction, we could construct 42 piles for the total 20 days by 2 machines. We could construct 2.1 piles per a day on the average, exceeded the original plan, 2 piles per a day. Fig.8 shows the planning cycle time based on construction test, and that of real construction(average).

4.2 Execution error

We decided that the allowable of execution error was one two-hundredth, based on execution guide of cast-in-place pile of Japanese Association of foundation Construction. When we excavated, we confirmed the accuracy of vertical direction by using together the inclinometer in excavation machine and the measurement by two transits from right-angled directions. When setting out was done for pitching of steel pile, we set the exclusive device for that with guide roller, with checking by transit. When pitching of steel pile was done, we checked the vertical by plumb bob, too. We secured the accuracy more than the stated that. While the strength of the set soil cement was solid more than the stated strength, 10 kgf/cm^2(design). The 4 week strength of core specimen from the pile head was 20 kgf/cm^2 on average.



Section	f(tf/m^2) Calculation	Test
②~⑤	13.0	19.5
⑤~⑦	7.0	8.9
⑦~⑧	15.0	23.5
⑧~⑨	6.0	19.1

Fig.7 The distribution of linear stress

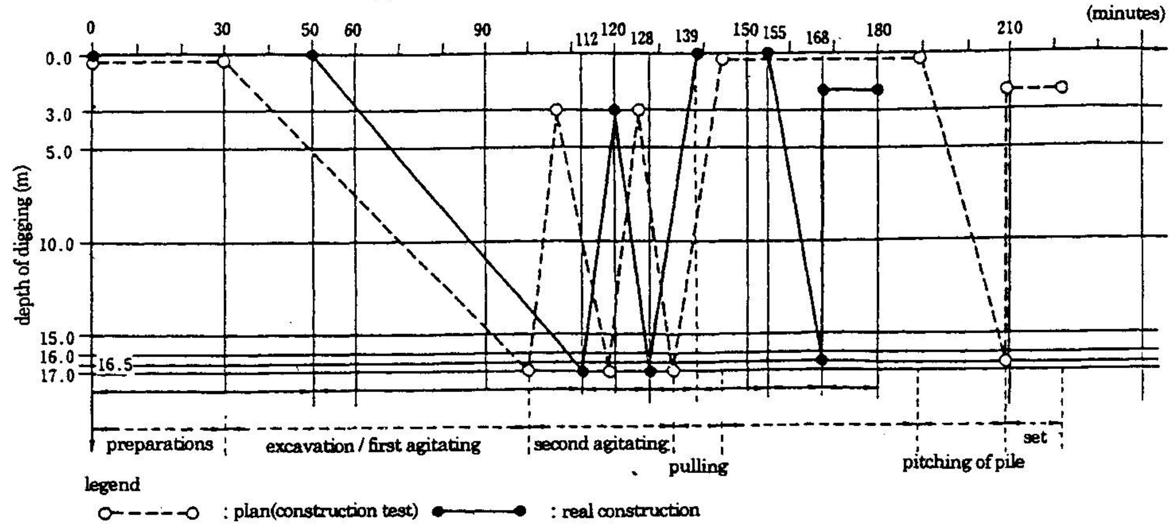


Fig.8 Cycle time

5 CONCLUSION

Soil cement and steel pile method is agitating and mixing ground and cement milk, needs little earth removal work, don't give industrial waste(slurry, etc). The method that this report showed, uses that method and rock auger method together. The cost is 20 or 30 percent higher than that of overall casing method. In this method, the material cost of steel pile with rib is high. This is one of matter for examination.

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Riprap Protection at Bridge Piers

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SUMMARY

The use of riprap to protect bridge piers against scour is considered. The four mechanisms of failure of riprap at bridge piers, i.e. shear failure, winnowing failure, edge failure and bed-form undermining, are discussed. A new design method for selecting riprap size is presented. The method, which is derived from many laboratory data, is based on the assumption that riprap failure occurs when the local scour depth at a riprap-protected pier exceeds 20% of the scour depth at the unprotected pier. The new method is compared with existing methods. The design of riprap protection at the Hutt Estuary Bridge in Wellington, New Zealand is discussed also. A physical model study was used as part of the design process for riprap protection at the Hutt Estuary Bridge.

1 INTRODUCTION

The most commonly employed method of protecting bridge piers against scour is the use of a layer of riprap around the piers. A U.S. field study reported 6000 cases of the use of riprap at bridge piers. The principle behind the use of riprap as a scour countermeasure is that large stones that are heavier than the river bed grains should be able to withstand the elevated shear stresses that occur around a bridge pier.

1.1 Riprap Failure Mechanisms

Four failure mechanisms of riprap layers at bridge piers can occur, as follows:

- Shear failure, where the riprap stones are entrained by the flow. Shear failure occurs where the armour units are unable to resist the hydrodynamic forces induced by the flow.
- Winnowing failure, where the finer underlying bed material is eroded through voids between the riprap stones under the action of turbulence and seepage flows. Winnowing is more likely to occur in sand-bed rivers than in coarser-bed materials. A filter layer, beneath the riprap layer, is often recommended to resist winnowing failure.
- Edge failure, where scouring at the periphery of the riprap layer undermines the armour stones. Riprap is vulnerable to edge failure if there is insufficient lateral extent of the protective layer.
- Bed-form undermining, where the riprap layer is undermined and settles with the migration past the pier of the trough of large dunes. Recent research (Melville, Lauchlan and Hadfield [10]; Lim and Chiew [9]; Parker, Toro-Escobar and Voigt [12]) indicates that bed-form undermining is the controlling failure mechanism at bridge piers founded in river beds subject to migration of dunes, especially sand-bed rivers. The settling of the riprap layer associated with bed-form undermining is significantly reduced with initial riprap placement at a depth of the order of the minimum expected trough level. With reduced settling of the riprap, the riprap layer remains reasonably intact. Conversely, a significant degree of settling is associated with destabilisation of the riprap layer due to the effects of the other failure mechanisms.

2 RIPRAP DESIGN FOR PIER PROTECTION

2.1 Stone Size

Most of the available methods for sizing riprap to protect bridge piers against scour can be expressed as follows:

$$\frac{d_{r50}}{y} = \frac{X}{\left(S_s - 1\right)^{\alpha}} Fr^{\beta} \tag{1}$$

where d_{r50} is the median size of the riprap stones; Fr is Froude Number of the approach mean flow = $V/(gy)^{0.5}$; V is mean velocity of flow; y is flow depth; g is acceleration of gravity; S_s is specific gravity of riprap stones; and X, α and β are coefficients. Amongst the various methods, α varies from 1 to 1.5, while β varies from 2 to 3. Some of the equations include factors for the effects of other influences on riprap stability, such as pier shape, pier size relative to riprap size and position of the pier in the channel.

The published equations are compared in Figure 1 over the range Fr = 0.2→0.6, with coefficients adopted for round-nose piers and S_s = 2.65. The methods of Chiew [5] and Parola [15] include an influence of the riprap size (d_{r50}) relative to the pier size (b). The methods by Austroads [1], Breusers et alia [3] and Croad [6] give very large riprap size, while that of Breusers and Raudkivi [4] gives very small riprap size. The Austroads [1] method is strongly dependent on the velocity factor, K_v, which varies with both the position of the pier in the channel and also whether the bridge is sited at a bend or otherwise. The values plotted in Figure 1 apply to a pier near the bank of a straight channel (K_v = 0.81) and a pier at the outside of a bend (K_v = 2.89). Chiew's [5] method can lead to both very large and very small riprap, depending on b/d_{r50}.

The remaining methods give riprap sizes within much narrower ranges, e.g. d_{r50}/y ranges from about 0.03→0.08 at Fr = 0.3, and from about 0.07→0.13 at Fr = 0.5, amongst the various methods. Lauchlan's

[8] method, which is plotted with the thick line, includes a parameter for the level of placement (Y_r) of the riprap within the sediment bed, which her study showed to be very significant. Given the lack of consistency amongst the methods, it is prudent to select a method that leads to conservatively large riprap relative to the other remaining methods. On this basis, the methods of Parola [14, 15], Richardson and Davis [18] and Lauchlan [8] are preferred. The method by Parola [14, 15], however, includes a significant influence of the pier size relative to the riprap size, b/d_{r50}, which is not present in the live-bed scour data measured by Lauchlan [8]. The expression presented by latter, given by the following equation, is recommended for selecting suitable riprap for bridge pier protection:

$$\frac{d_{r50}}{y} = 0.3S_f\left(1-\frac{Y_r}{y}\right)^{2.75} Fr^{1.2} \tag{2}$$

where S_f is factor of safety, with a minimum recommended value of 1.1. Equation (2) is based on laboratory data using a failure criterion whereby the riprap was adjudged to have failed when the local scour depth at a protected bridge pier exceeded 20% of that at the same pier without riprap protection. The relation in equation (2) for the effect of placement level, i.e. $(1-Y_r/y)^{2.75}$, was determined also from laboratory data using the same failure criterion. Curves for different values of Y_r, based on equation (2), are given in Figure 2. It is demonstrated in Figure 2 that equation (2), with $Y_r = 0$ (i.e. riprap laid at bed level), is an envelope to many other published data. It should be noted that different failure criteria were applied amongst the various sets of data plotted, including failure defined by complete disintegration of the riprap. The "20% scour-depth" criterion adopted by Lauchlan [8] is the most rigorous criterion, consistent with her equation enveloping the other data.

2.2 Riprap Placement

Other important factors relating to riprap design are the thickness (t_r), horizontal extent, and placement level (Y_r) of the riprap layer. Figure 3 shows recommendations for these factors, based principally on recent studies by Lim and Chiew [9] and Lauchlan [8]. Parker, Toro-Escobar and Voigt [12] have shown that the use of a synthetic filter (or geotextile), placed below the riprap, is advantageous because the effects of winnowing failure are minimised. Their experiments demonstrate, however, that the synthetic filter should have lateral extent limited to about 75% of that of the riprap. With a full-coverage geotextile, edge failure of the riprap stones can result in roll-up of the edges of the geotextile leading to subsequent failure of the riprap layer. Typical recommendations for riprap grading, e.g. Croad [6], are as follows:

$$0.5d_{r\max} < d_{r50} < 2d_{r15} \tag{3}$$

where d_{rmax} is the largest stone size, and d_{r15} is the stone size for which 15% of the stones are finer by weight.

3 HUTT ESTUARY BRIDGE

3.1 Background

The design of riprap protection at the piers of the Hutt Estuary Bridge, near Wellington, New Zealand, is discussed in the following. The 5-span bridge is 179 m long and is sited about 1 km from the river mouth in Wellington Harbour. The bridge has a history of scour problems due to significant bed degradation, which occurred until recently, the degradation arising from uncontrolled gravel mining at several sites upstream. The piers are rectangular (1.1 m wide, 16.4 m long) with slab footings, the footings having been encased with steel caissons (5.5 m wide, 16.4 m long). The approximately 7.6 m deep caissons were installed to protect the piers against the bed degradation. Bed material is a coarse sand with median size d_{50} = 2 mm. Flood levels are controlled by associated sea levels at the river mouth. For the 1% AEP design flood, the flood level is 1.86 m above mean sea level (MSL). The general scour depth under the design flood was estimated, using the Blench [2] equation, at 4.1 m below MSL or 5.96 m below flood level. The corresponding flow velocity is V = 3.6 m/s. With a flow depth of y = 5.96 m following general scour, the tops of the caissons would be about 2.3 m above the scoured bed level. The method by Melville and Raudkivi [11] for local scour depth, predicts a total scoured depth of 13.1 m below flood level. On the basis of these calculations, it was recognised that the bridge would be seriously endangered in the design flood. A cross-section of a pier, showing the calculated scour depths, is given in Figure 4.

3.2 Model Study

A model study was undertaken to assess the requirements for riprap protection at the piers. The model comprised one pier, built to a scale of 1:20, and was tested in a 1.52 m wide laboratory flume using Froudian scaled flow parameters and a uniform sand of median size 0.8 mm. During preliminary testing, the model results were found to confirm the local scour depth calculations for the unprotected pier.

Riprap was selected based on the Lauchlan [8] equation (2) with $Y_r = 0$. For $V = 3.6$ m/s and $y = 5.96$ m, $d_{r50p} = 724$ mm, equivalent to a model size of 36.2 mm. Two sizes of riprap were used in the model, having median sizes, $d_{r50m} =$ 35 mm and 28 mm, and grading satisfying the criteria of equation (3). Note that subscripts 'p' and 'm' refer to prototype and model, respectively. The riprap was placed according to the recommendations in Figure 3 (end radii = 1.5b, width = 3b), with the top surface flush with the undisturbed bed. Tests were conducted with the pier both aligned with the flow and also skewed at angle $\theta = 10°$ to the flow. Two thickness arrangements were tested. One arrangement featured $t_r = 2d_{r50}$ throughout the layer. For the other arrangement, the thickness of the riprap layer was increased to $3d_{r50}$ for the section of riprap upstream of the pier where it was observed that the worst scour occurred.

The results are shown in Table 1, which gives the percentage reduction in local scour depth, from that at the unprotected pier, afforded by the different riprap arrangements.

Test	d_{r50} (mm)	t_r	θ (°)	Percentage Scour Reduction (%)
1	35	$2d_{r50}$ throughout	0	71
2	35	$3d_{r50}$ upstr./ $2d_{r50}$ elsewhere	0	97
3	35	$2d_{r50}$ throughout	10	77
4	35	$3d_{r50}$ upstr./ $2d_{r50}$ elsewhere	10	88
5	28	$3d_{r50}$ upstr./ $2d_{r50}$ elsewhere	10	76

Table 1 Results for Model Tests of Riprap Protection at Hutt Estuary Bridge

For all tests, the scour reduction was significant. The riprap configuration for Test 4 was recommended. The riprap comprises a 1400 mm thick layer of 700 mm riprap thickened to 2100 mm upstream of the pier. The local scour depth is reduced 88% when the flow is skewed 10° to the axis of the pier. The recommended riprap configuration is illustrated in Figure 4.

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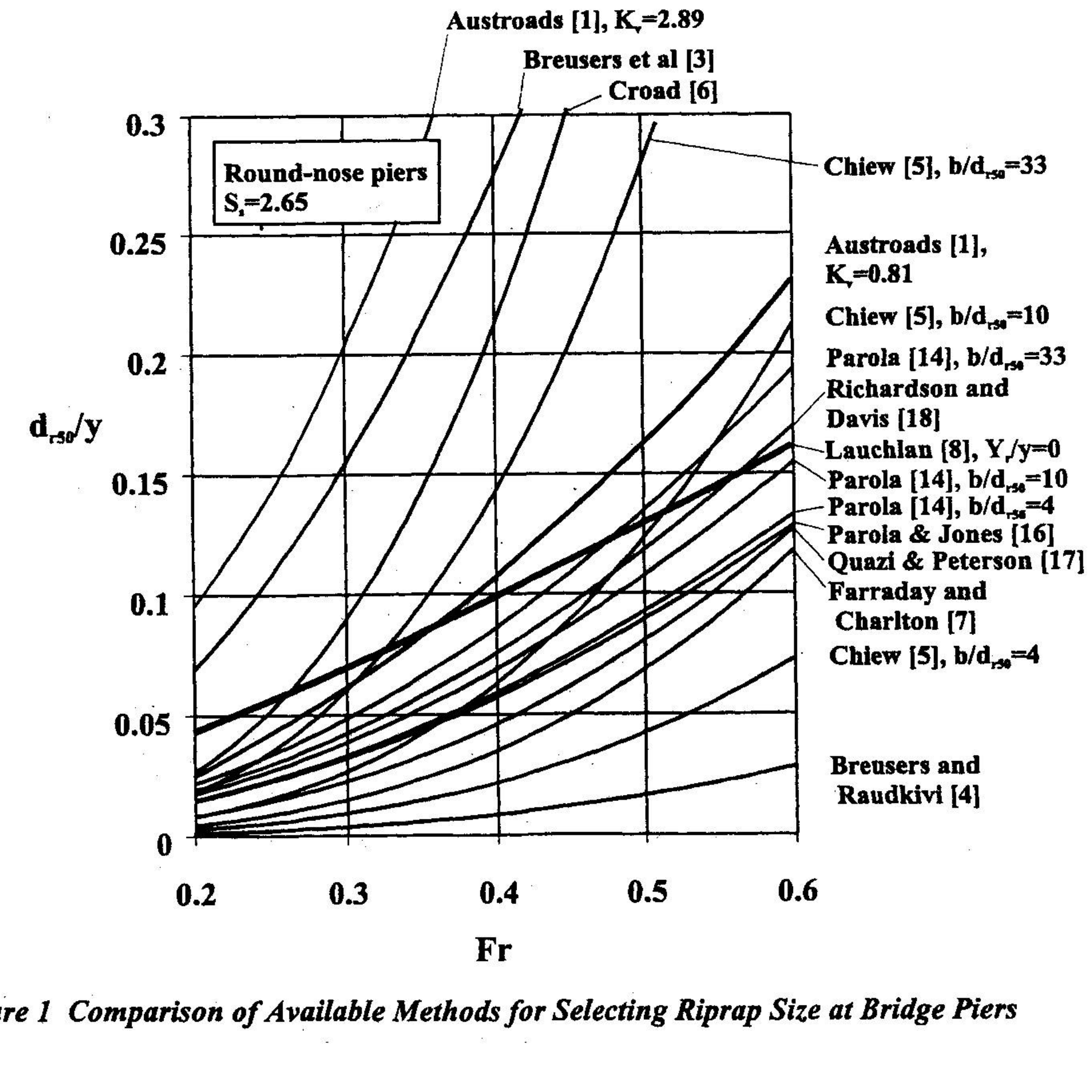

Figure 1 Comparison of Available Methods for Selecting Riprap Size at Bridge Piers

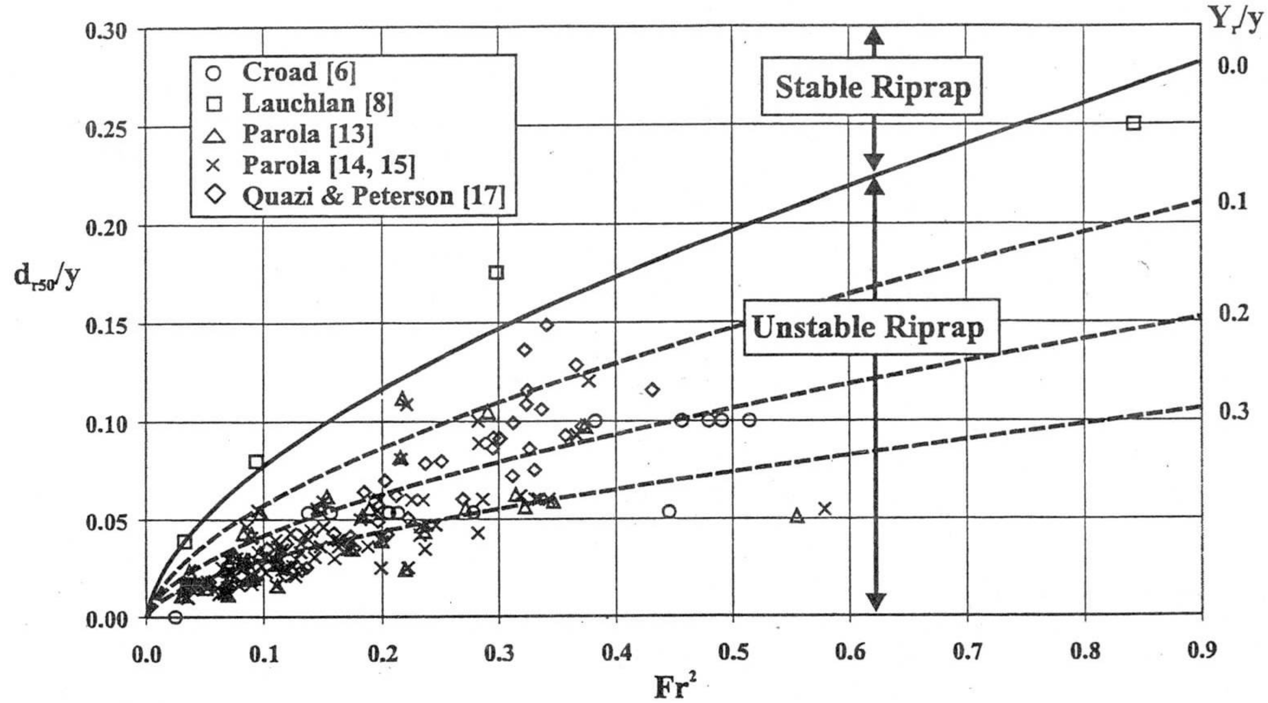

Figure 2 Design Curves for Riprap Size by Lauchlan [8]

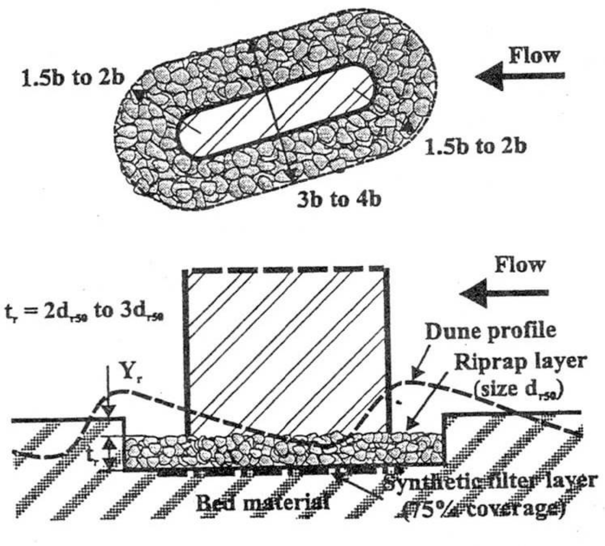

Figure 3 Riprap Placement Recommendations

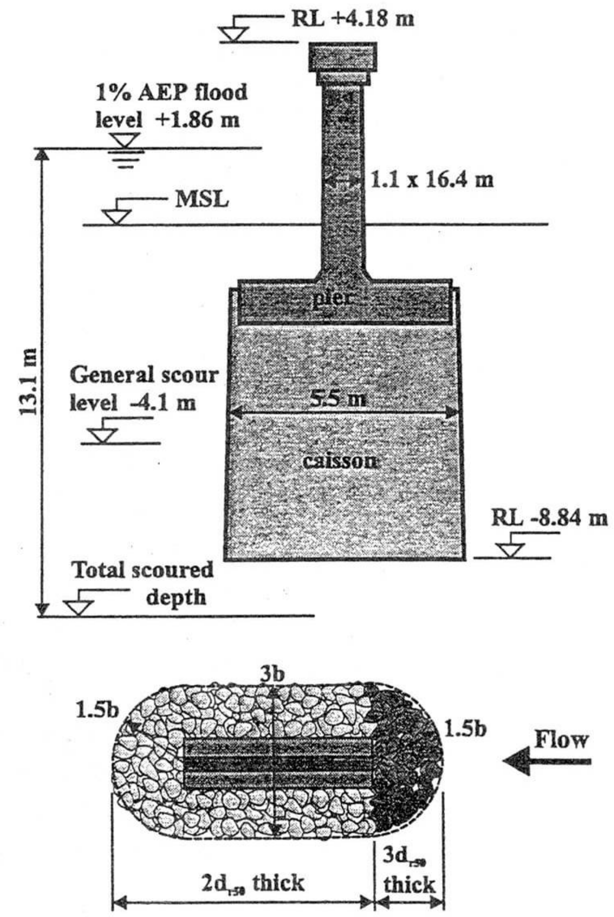

Figure 4 Riprap Design for Hutt Estuary Bridge

New Zealand Case Studies of Scour at Bridge Foundations

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SUMMARY

Three case studies of scour at New Zealand bridges are presented to illustrate examples of components of scour which need to be considered in bridge foundation design. The Ashburton River Road Bridge was threatened by scour arising from a combination of progressive degradation, local scour, and debris accumulation effects. The Bulls Road Bridge failed principally as the result of a combination of progressive degradation, scour at a bend, local scour for oblique flows, debris accumulation effects, and lateral loading for the failed pier. The Waitangitaona River Road Bridge failed as the result of the combination of scour at the convergence of flow paths (scour at a confluence), contraction scour, local scour for oblique flows, and debris accumulation effects. Various forms of measures proposed and adopted for remediation and prevention of scour in these cases are presented, where these measures include modified pier designs, riprap aprons, riprap weirs, and riprap-lined guidebanks.

1 INTRODUCTION

Sediment movement or erosion around bridge foundations, referred to as bridge scour, can result in damage to the bridge structure. The types of scour that can occur at a bridge crossing are typically referred to as ***general scour***, ***contraction scour*** and ***local scour***. ***Progressive degradation*** or ***aggradation***, ***thalweg effects***, ***scour at a bend***, ***scour at a confluence***, and ***bed-form migration effects*** are all components of general scour which act to influence riverbed elevations. General scour components influencing the planform of the river include ***shifts in channel bends***, ***meander migration***, ***shifts in braids or anabranches*** within the channel, and also ***lateral bank erosion due to channel widening***. Whereas general scour occurs irrespective of the existence of the bridge, contraction and local scour arise principally due to the presence of the bridge, contraction scour occurring as a result of the flow being constricted by the bridge foundations. Local scour is caused by the direct interference of the bridge foundations (abutments and piers) with the flow. The effects of ***debris rafting*** at a bridge site further magnify any erosion around the foundations and also any lateral and vertical forces on the bridge due to debris and sediment loads. At a particular bridge crossing, any or all of the different types of scour may occur simultaneously, the total scour being the sum of the individual scour components.

Three cases of bridge scour damage that have occurred within New Zealand are presented herein. In each case, the bridge and the site are described, the damage process is outlined, and any adopted remedial measures are discussed. The cases presented have been selected to highlight both the various vertical scour components occurring for bridge sites, and also a range of appropriate remedial measures for endangered bridge structures.

2 ASHBURTON RIVER ROAD BRIDGE

This road bridge over the Ashburton River is a 340 m long, two lane, reinforced concrete structure that was built in 1931. The bridge comprises 31 slab-type piers (Figure 1), about 25 of which lie within the active river channel. Each pier is founded on seven 400 mm reinforced concrete octagonal piles, these piles having been driven to a relatively uniform depth of between 6.5 m and 6.7 m below the underside of the pile caps. The original average bed level was measured at 0.2 m below the underside of the pile caps.

The Ashburton River in the vicinity of the bridge site is about 280 m wide, is straight, uniform in slope and width, and is bounded by trees and straight stopbanks. The channel is braided and there is evidence of active bed movement. The bed material is well-graded gravel. The similar amount of debris caught at each of the piers confirms that flood flows are uniform across the channel.

Over the life of the bridge, various stopbanking, river-clearing works and gravel-extraction works have taken place over extensive lengths of the river, upstream and downstream of the bridge. The extraction of gravel from the river has been and continues to be controlled.

General scour exacerbated by gravel extraction from the river upstream and downstream of the bridge has resulted in a gradual lowering of the bed level at the bridge site (Figure 1). The average bed level lowered approximately 0.9 m from 1937 to 1994, each of the piers within the river channel having exposed piles. In 1978, the average bed level was measured at a level of 1.7 m below the underside of the pile cap. The bridge at this time was concluded to be extremely vulnerable to scour damage by pier undermining arising from the progressive degradation of the bed. Concern for the stability of the bridge was accentuated by the shallowness of the foundation piles and also debris rafts forming at the site increasing the potential for local scour pier undermining and bridge damage or failure in significant floods.

Rock aprons were constructed in 1979 around each of the piers within the active channel (Figure 1). Each apron measures 5 m wide, 15 m long and 1.6 m thick. Rock riprap of a median size d_{50} of 0.5 m was used to construct the aprons, with the upper surface of each apron located beneath the riverbed surface and approximately 2 m below the base of the pile cap. Aprons act to reduce the scour potential at piers both by armouring the bed against local scour due to local hydraulic vortices, and also by protecting against general scour by dropping at the apron extremities as this scour develops. Aprons nevertheless cannot provide total assurance against scouring, particularly for ongoing general degradation. Inspection in 1994 indicated the rock apron to be exposed at one pier only. Bed levels in the river channel continue to be regularly monitored.

The bridge has to date performed satisfactorily in the large floods over its life, even with the bed lower than its present level. Possible additional future scour damage mitigation options which have been identified include underpinning of the piers and the construction of a weir immediately downstream of the bridge, with the weir crest level no lower than the underside of the pier caps. The usual form of weir adopted is a rock riprap structure comprising a crest and downstream apron. Such weirs have been used very successfully in the West Coast and Canterbury areas of New Zealand to control degrading river channels. At the Ashburton River Road Bridge site, the weir would significantly affect fish migration, riverbed levels, and stopbank effectiveness upstream of the bridge, and also bed levels immediately downstream of the weir, which would degrade.

3 BULLS ROAD BRIDGE

This two-lane structure over the Rangitikei River opened in 1949 and consisted of 17 spans of 26.8 m length, with two end spans of 20 m length. Alternate spans had a cantilever suspended span in mid-section. Each pier was a reinforced concrete slab type and was founded on two rows of six vertical 0.4 m octagonal reinforced concrete piles. The piles had been difficult to drive, anecdotal evidence suggesting that a number of piles had fractured upon being driven. The piles at Piers P and Q extended approximately 8.75-9.5 m below the base of the pile cap. The piles were founded in a shingle (gravel) surface stratum, extending to about 4.7 m below the base of the pile cap, underlain by a thin papa (mudstone) layer of about 0.9 m thickness, and then a layer of a fine black sand.

In 1947, the river was significantly braided (Figure 2), with numerous channels upstream of the bridge site. Bed levels were relatively uniform over the width of the cross-section. With gravel extraction occurring from 1949 upstream and downstream of the bridge site, the number of braided channel branches reduced to 1-2 main channels meandering within the wider channel (Figure 2). The new flow channels were deeper and narrower than the previous channels. Such developments arising from gravel extraction from a river can be forecasted based on qualitative methodologies describing channel development. River control works were developed over the period from 1949 to 1973 to protect the outsides of the meander bends upstream of the bridge. A high terrace that is relatively erosion-resistant defines the true-left bank of the wider channel. The river meanders reinforced by the bend protection works immediately upstream of the bridge resulted in the main flow channel deflecting off the true-left terrace just upstream of the bridge and passing underneath the end of the bridge at an angle of about 55^{o} to the bridge centreline in the early 1970s (Figure 2).

In conjunction with the gravel extraction, mean and minimum bed levels at the bridge site fell relatively steadily over the period from 1945 to 1972. Anecdotal evidence based on cross-section measurements indicates the existence of about 6 m depth of scour at Pier Q about one month prior to failure. With the deepening of the main flow channel under the bridge, an old timber pier was exposed immediately downstream of Pier Q in 1971-1972 (Figure 3).

On June 15, 1973, failure of the bridge occurred on the falling limb of a flood hydrograph estimated as approximating an annual flood event. The flood flows oblique to Pier Q (Figure 3) are estimated to have eroded through the thin papa layer and exposed the underlying fine river sand to the flow. The oblique angle of the flows to the slab-type pier and the increase in material erodibility would then have increased the depth and rate of development of the local scour at this pier. Flow between Pier Q and the exposed debris-enlarged old timber pier immediately downstream (Figure 3) would also have induced increased local flow velocities around Pier Q, further increasing local scour rates and depths. The reduced fixity at the base of the piles as the scour developed, combined with the flows oblique to the pier exerting lateral pressures on the exposed piles, resulted in hinging occurring both at the base of the piles and also at the underside of the pile cap for Pier Q (Figure 3). The collapse of Pier Q together with the end of the bridge deck connecting Piers P and Q also caused rotation of Pier P, with the suspended spans adjacent to Piers P and Q falling into the river (Figure 3).

Each of Piers P and Q was replaced by a single column of 1.83 m diameter supported by a cylinder of 2.13 m diameter. The respective cylinders for Piers P and Q extend to 17.1 m and 18.9 m below the level of the base of the old pile cap. Piers R, S and H to O were also underpinned by 2 cylinders of 1.7 m diameter spaced at 12.95 m centres, the cylinders extending down to 17.1 m below the level of the underside of the original pile cap. To control bed levels, strict control has also been introduced on gravel extraction. It is of interest to note that from 1992 to 1997, the main river channel passed principally between Piers J-M (Figure 2).

4 WAITANGITAONA RIVER ROAD BRIDGE

This bridge across the Waitangitaona River is a 149 m long (6 spans of 24.7 m) two-lane carriageway, which was built in 1968 to replace a bridge that needed to be raised owing to channel aggradation (resulting from a major slip upstream in the 1920s). The new bridge was supported by reinforced concrete abutments and slab piers, each pier being supported by a row of nine 380 mm square blunt-ended prestressed concrete piles, all but the central pile being raked at 1:8 (H:V) along the line of the pier. The bridge was initially designed for a scour depth of 1.8 m below minimum bed level, piles for the piers being driven to 9.3 m below minimum bed level. The channel bed is composed of surface gravels and cobbles with relatively minor amounts of sands and silts, and a substrata of medium to dense gravels.

The bridge, which spans a channel forming a sudden contraction for flood flows, is located at the exit from a wide channel containing a braided river at low flows (Figure 4). A shift and shortening in the river course downstream of the bridge resulted in rapid bed degradation at the bridge site during construction of the bridge in 1967. This degradation and later flooding resulted in piles extending to only 6.8 m below the minimum bed level. The angle of flow attack on the slab piers also increased at this time. The channel, which was relatively flat between the five central piers, was subsequently essentially stable in alignment and cross-section.

On 12 March, 1982, a flood occurred for which the peak flow was estimated to be 700 m^3/s (approximately 82% of the design flood flow of 850 m^3/s). The bridge had previously withstood a flood flow in June 1967 estimated at 700 m^3/s. Debris, including trees up to 20 m long and 1.2 m in diameter, were noted in the flow and adjacent to the piers as the flood receded, the skewed approach flows holding the debris against the sides of the exposed piles of the piers during the flood event.

For the 1982 flood, it appears that the two approach embankments concentrated flow between Piers D and F at an angle of about 30° to the face of Pier E (Figure 4). The flood flows at a skewed angle to the pier, exacerbated by debris accumulation effects, resulted in increased lateral loading and increased scour depths for Pier E. Pier E was consequently scoured out, with the associated loss of the two adjoining deck spans (Figure 4). The remaining sections of the bridge were undamaged. Scour holes at the piers were noted to be infilled as the flood receded.

New guidebanks extending into the channel were subsequently established through the bridge site (Figure 4). A 1.5 m thick riprap layer at a slope of 2:1 (H:V), extended by a horizontal riprap toe apron 2 m thick with its upper surface at the existing bed level, was used to protect existing piers located within the new guidebanks. For these piers, the existing piles were also supplemented with three steel H-piles at each end of the pile cap, these additional piles extending to 13 m below the design bed level. The riprap utilised consisted of rocks of $d_{50} = 750$ mm. Each pier remaining in the revised stream channel was underpinned with two cylinders of 1.5 m diameter at 11 m centres, each cylinder connected to the existing pier via a 1.0 m octagonal column above the design bed level. The cylinder foundations were driven to 13 m below the bed. The existing piled foundations and mass concrete skirt for each of these piers in the revised stream channel were removed along with the base of the slab pier to 2m above the design flood levels, this level allowing some clearance to minimise any debris accumulations at the piers.

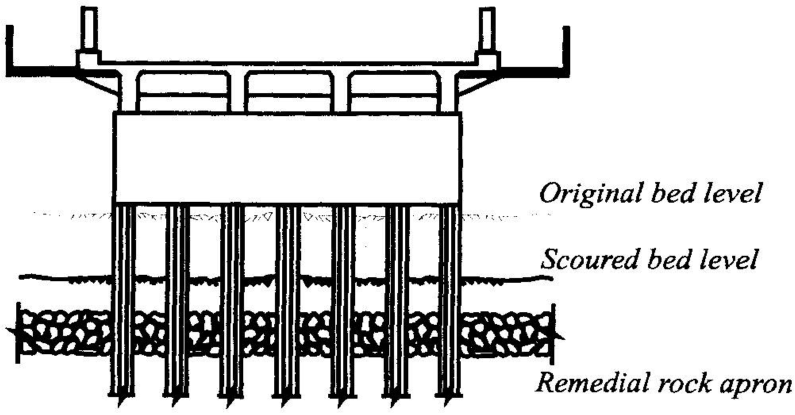

Figure 1 Typical Side Elevation of a Foundation and Remedial Works for the Ashburton River Road Bridge

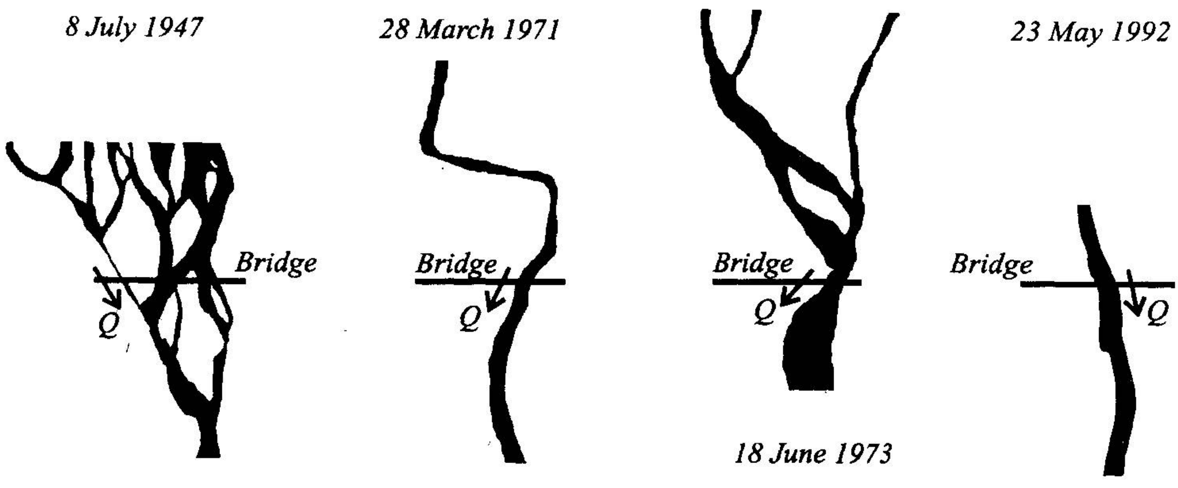

Figure 2 Development of the Principal Flow Channels of the Rangitikei River at the Bulls Road Bridge

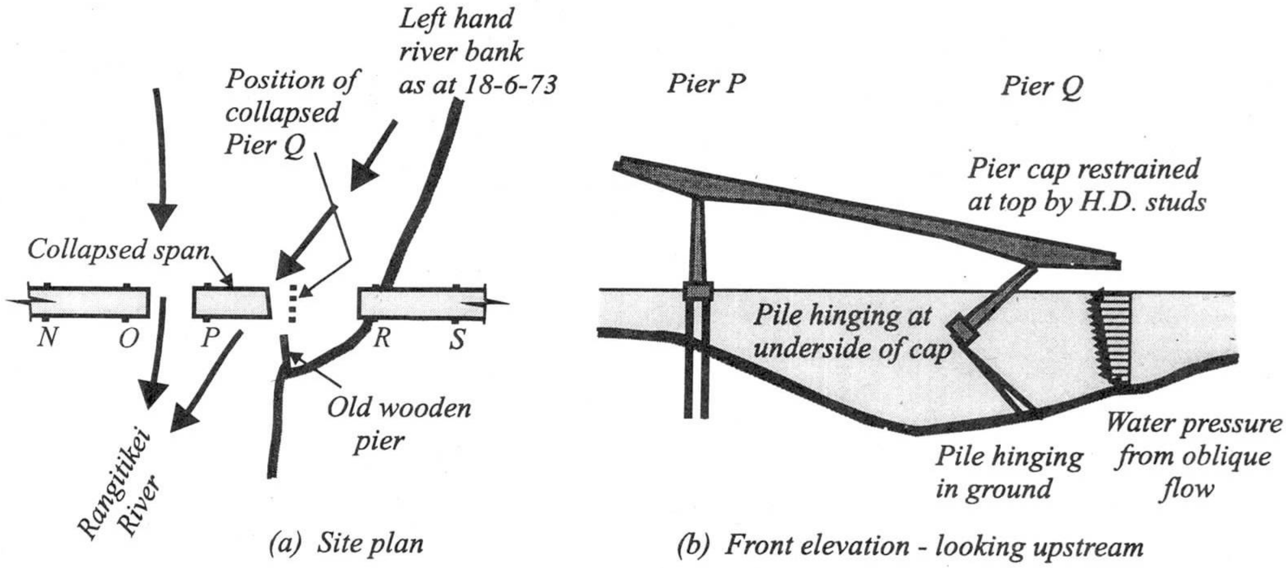

Figure 3 The Bulls Road Bridge Failure

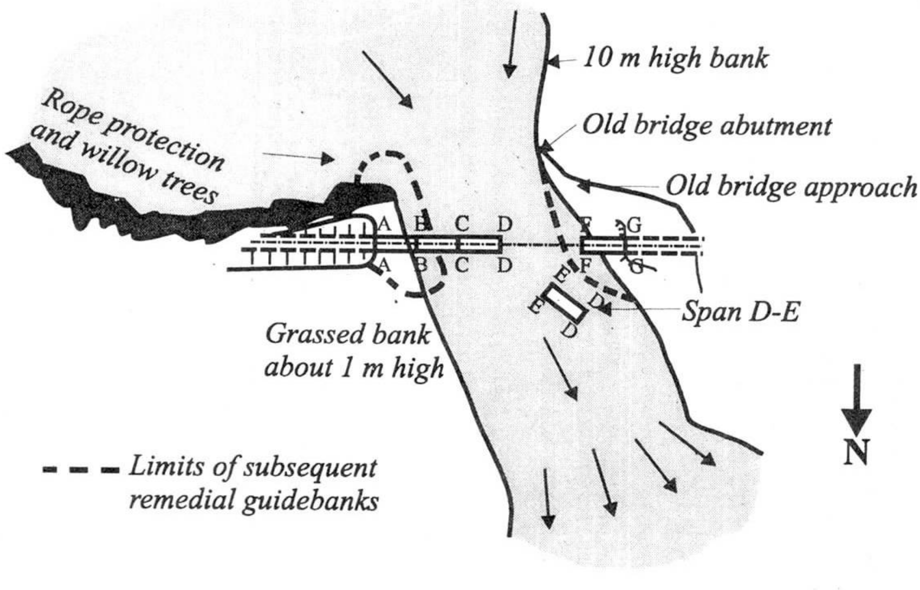

Figure 4 Schematic Site Plan of the Waitangitaona River Road Bridge Failure

FOUNDATION DESIGN AND CONSTRUCTION OF RAIL-CUM-ROAD BRIDGE ACROSS RIVER DAMODAR NEAR MEJIA

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SUMMARY

This paper highlights the special features of planning , design and construction of the foundations for the road-cum-rail bridge across river Damodar along the alignment of the Raniganj-Mejia dedicated railway line for the Thermal Power Plant of Damodar Valley Corporation. This bridge provides a vital link for rail as also road transport between the Burdwan and Bankura districts of West Bengal, which earlier had inadequate connectors. The experiences on this bridge are being effectively utilized in other bridges being built in eastern region and north-eastern region, as this was the first major road bridge built on pile foundation in this part of the country.

1. PREFACE

Damodar Valley Corporation (DVC) set up a captive railway system for transportation of coal by rail to their Thermal Power Plant at Mejia in the district of Bankura of West Bengal. This railway line is designed to carry 7,000 tonnes of coal daily for meeting the requirements of the 630 MW power plant. The line is a major crossing on the river Damodar near Raniganj.

The railway bridge was initially envisaged to have 9 nos. of through type steel girders of clear span 76.2 m and two shore spans of 30.5 m. This arrangement of girders had been recommended by the consultant appointed by DVC for preparation of project report for the captive railway line[1].

2. NECESSITY OF RAIL-CUM-ROAD BRIDGE

The river divides the neighbouring districts of Burdwan & Bankura in West Bengal over long stretches during the flood season and, except the Durgapur barrage, there is no direct road link between these two neighbouring districts. In deference to the popular demand for additional link between the two neighbouring districts by road over the river, it was decided to introduce a 2-lane road superstructure of National Highway standard on the common foundations with the rail bridge.

3. SPAN ARRANGEMENT

The span arrangement of the rail-cum-road bridge was adopted as 15 x 48.5m + 2 x 33.062m after comparing the cost economy for various span arrangements, taking into account the varying costs of rail and road superstructure and the common foundation.

The superstructure was made structurally isolated with the roadway deck and railway track, located side by side on common substructure.

The superstructure for the railway has 15 nos. 45.7 m (clear span) standard through type steel truss span as per standard drawings adopted by the Railways with two shore spans of 30.5m (clear span) of similar nature. The roadway superstructure consists of three girder, precast, post-tensioned concrete T-beams with composite R.C. deck slab, carrying 7.5m-wide carriageway and 2m-wide footpath on one side only.

4. DESIGN CONSIDERATIONS

4.1 Hydraulic parameters

As a prelude to the construction of a permanent bridge, the hydrulic parameters for the bridge were first established. The location of the bridge site is approximately 15 km upstream of the Durgapur barrage in the district of Burdwan, West Bengal. The value of the discharge for this bridge was considered to be 18400 cumec[2]. , which is same as the design flood discharge of the Durgapur barrage.

The effective width of waterway for the above discharge works out to be around 656 m, which is less than the effective width of waterway provided from topographical considerations.

4.2 Soil investigation & profile

Soil investigation work for the foundation along the same alignment was carried out ahead of the decision to have a rail-cum-road bridge. Soil investigation had, therefore, been carried out at the locations of two abutments . and the ten pier locations as per the originally envisaged scheme, i.e. for 2 x 33 m + 9 x 80.1 m span pier arrangement. The bore hole positions, therefore, did not match with the new span arrangement, but a clear picture of soil profile distribution along the alignment was established from the soil investigation report, before undertaking the design of the foundations for this bridge.

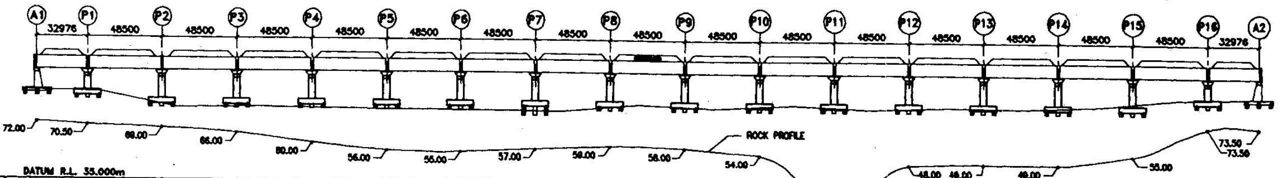
During construction, boreholes were installed at each pier location, which the substrata confirmed the existence of sandstone at designed levels overtopped by loose-to-medium dense sand or silty sand with traces of gravels. It was seen from the generalized soil profile that the rock slopes down towards the centre of the river from either abutment end and no rock layer could be struck at the central region (pier locations 11, 12, 13 & 14) where boring was continued as much as 35.00 m below the bed level without reaching rock strata. This indicates the possible existence of a geological rift along the river alignment.

5. SELECTION OF FOUNDATION

The availability of rock at a depth of around 15 m average from the bed level and existence of high groundwater level eliminated the adoption of open foundations. Acceptable options for foundation were

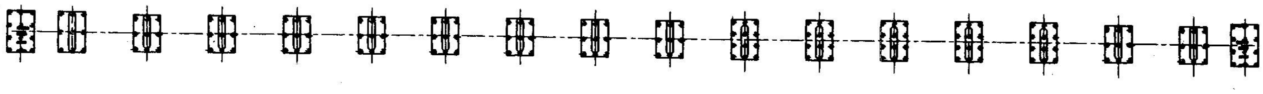
i) Caissons or well foundation

ii) Large diameter pile foundation



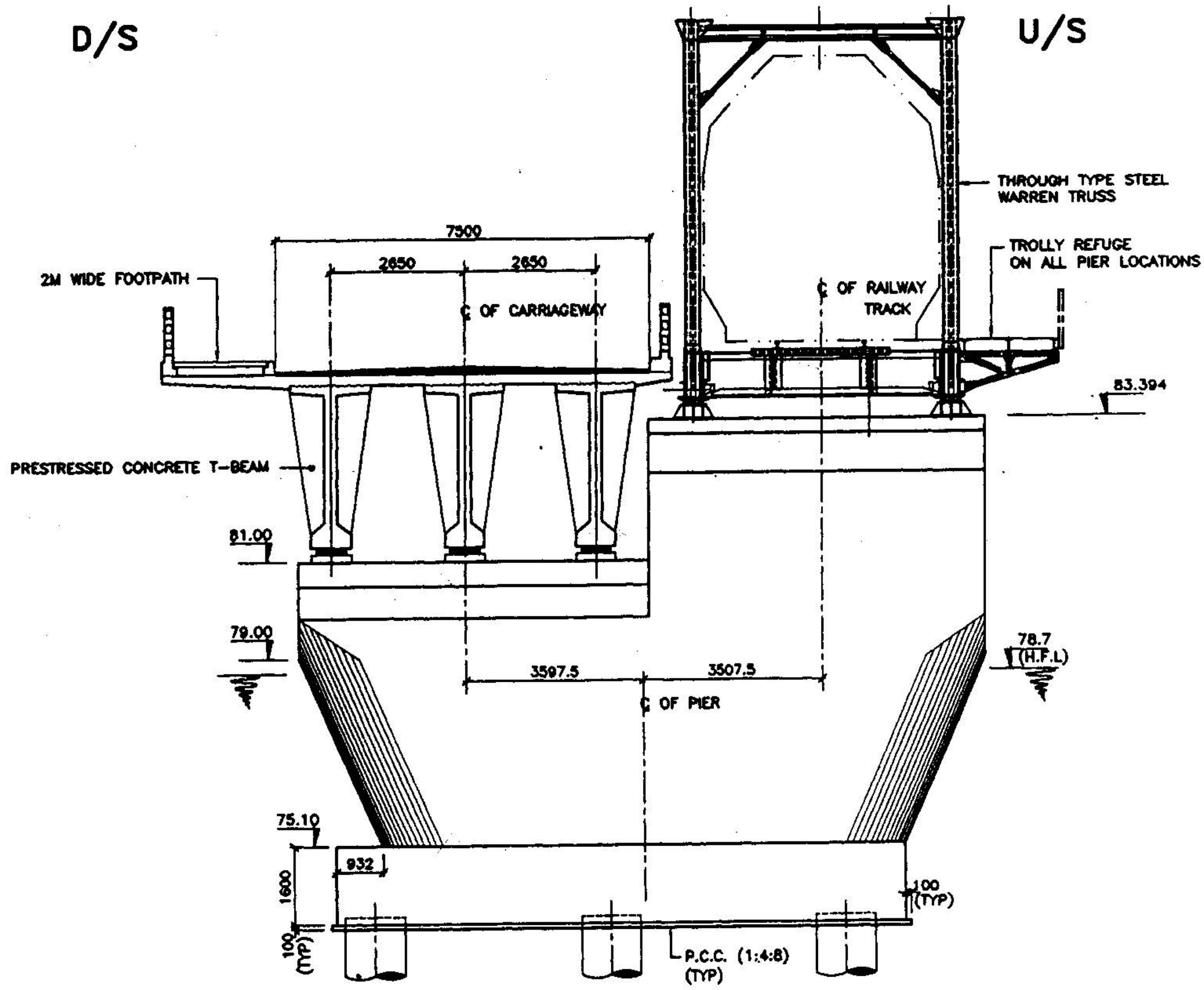
BED LVL (m)	77.580		77.180	72.500	72.740	72.490	72.320	72.100	72.000	74.100	73.150	74.500	73.250	74.000	73.850	74.770	77.250		77.580
CHAINAGE (m)	700.000	703.274 (FACE OF DIRT WALL)	750.000	800.000	850.000	900.000	950.000	0.000	50.000	100.000	150.000	200.000	250.000	300.000	350.000	400.000	450.000	(FACE OF DIRT WALL) 496.726	500.000

GENERAL ARRANGEMENT AND GEOTECHNICAL PROFILE OF RAIL-CUM-ROAD BRIDGE ACROSS RIVER DAMODAR

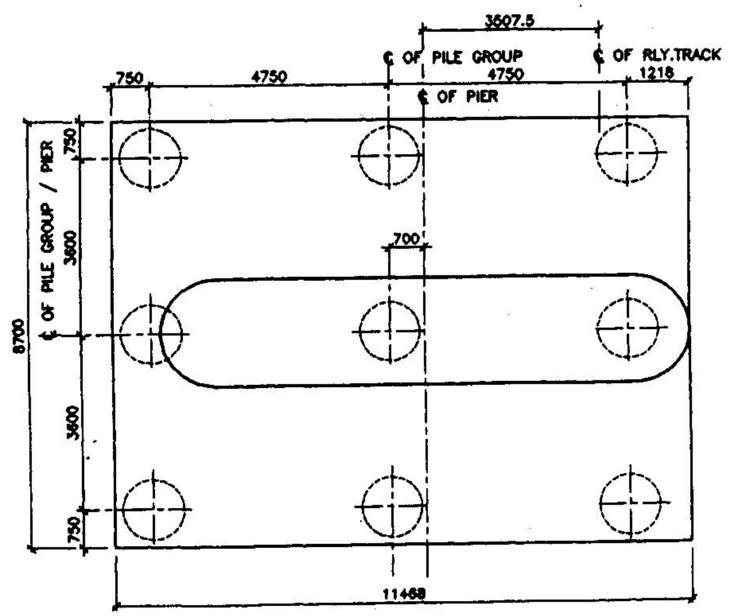


PLAN AT PILECAP LEVEL

FIGURE :1



TYPICAL CROSS SECTION OF RAIL-CUM-ROAD BRIDGE OVER RIVER DAMODAR NEAR MEJIA



PLAN OF PILECAP

FIGURE:2

Both these systems have inherent functional characteristics which were closely weighed before making the final selection.

The caisson or well foundation provides welcome stability when the foundation is subjected to large lateral forces at a high leverage. Construction of the well foundation, however, is time consuming and involves considerable preparatory time. Sinking of well is an uncertain process, particularly in bouldery layers.

Construction of large-diameter bored cast-in-situ pile is a relatively assured process. Large depths can be achieved with bored cast-in-situ pile in almost all soil strata, including bouldery layers. With the adoption of large-diameter piles it is possible to overcome the limitations of achieving higher load carrying capacity, both vertically and laterally. In Damodar river, the rock qualities are heterogeneous in nature and there can be observed a rapid change of rock levels within a short distance. Adoption of end-bearing bored cast-in-situ pile was the preferred foundation element for this bridge, after comparing installation cost and time cost.

6. SUBSTRUCTURE

6.1 Pier

Wall type R.C.C. pier was adopted for this bridge for supporting the isolated superstructures for both railway and roadway. The railway formation level at the bridge location was finalized keeping in view the overall economy of the project. The roadway formation level was fixed at the same level as railways to reduce approach viaducts. The railway traffic moves on deck system supported on the bottom chord of trussed girders, whereas the roadway live load moves on the top of T-beam superstructure. This resulted in a stepped pier cap as seen in figure-2.

The pier was designed to withstand the permanent dead load and a wide variation of live loads in addition to the longitudinal forces due to braking/traction, water current and wind. The variations of the live load combination which were considered are as follows:

Case-1	One side railway span loaded
Case-2	Both side railway span loaded
Case-3	Both side railway span loaded + One side roadway span loaded
Case-4	Both side railway span loaded + Both side roadway span loaded
Case-5	One side railway span loaded + Both side roadway span loaded
Case-6	Both side roadway span loaded
Case-7	One side roadway span loaded
Case-8	One side railway span loaded + One side roadway span loaded

It is important to note that while the deadweight of the roadway span is much higher than that for the railway span, the railway live load effect is significantly greater than that of the roadway live load. The combined effect of these loads results in wide variations of stresses on the foundation elements as can be seen from the figure-3. The pier configuration was made eccentric with respect to the geometrical centre of the two superstructures and the pile groups for partially compensating for this loading pattern

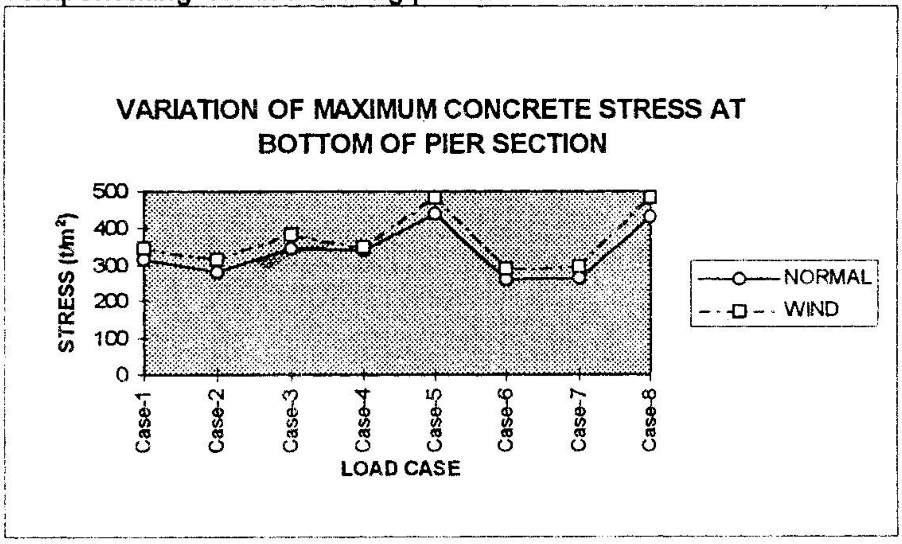


figure-3

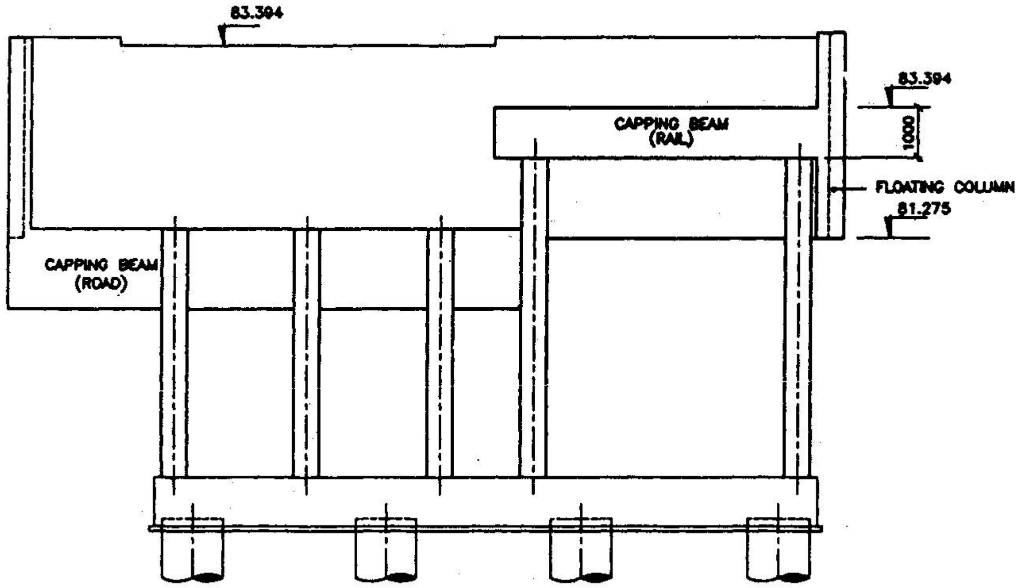
6.2 Abutment

Spill through type R.C. abutment with rectangular RC column was provided for this bridge with cantilever type wing wall and dirt wall (ref. figure-4). The level of the cap of abutment followed the same arrangement as for the pier. For reducing the abutment cap section, rectangular RC columns have been provided under each bearing location. The distribution of lateral forces on each column has been made on the basis of stiffness coefficient method taking into account the rigidity of columns. The column sections have subsequently been checked by 3-D frame analysis, considering the abutment as a framed structure subjected to vertical loads, lateral loads and moments. The bending moment at the base of columns compared and it was noted that the value obtained from 3-D frame analysis were 10% lower for columns under road superstructure and 18% lower for columns under rail superstructure. The design of sections, however, were prepared considering the higher values.

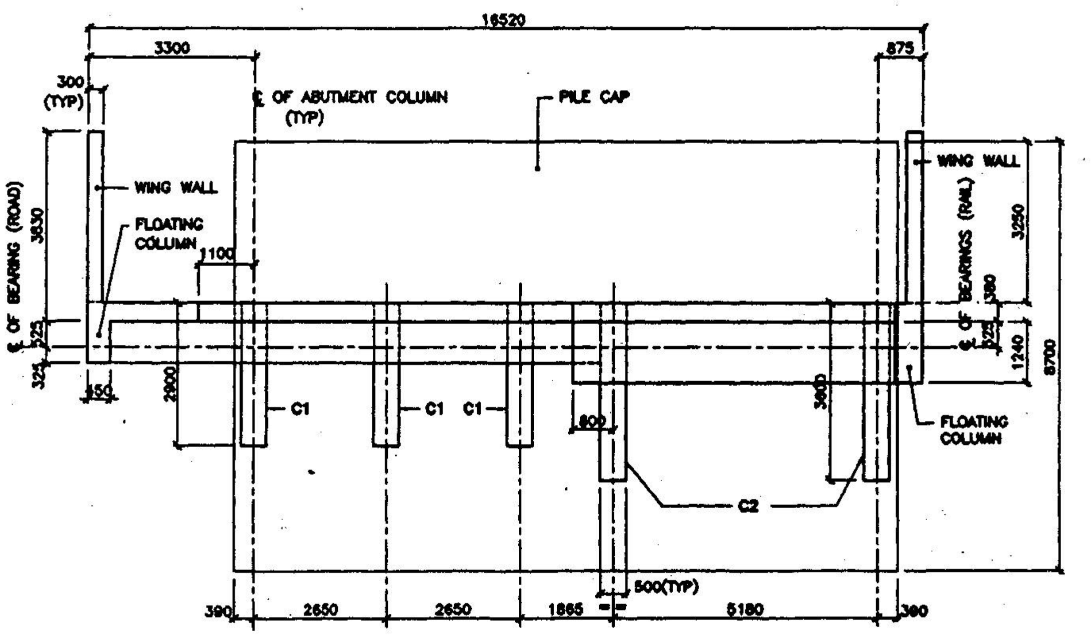
7. PILE FOUNDATION

Large-diameter piles of 1,200 mm dia have been used after examining the relative economy with possible alternative sizes. Considering the generalized soil profile of the river as reflected in figure-1, the pile groups are divided under two heads.

Group-A	:	Piles resting on rock
Group-B	:	Piles embedded in sandy strata



ELEVATION OF ABUTMENT



PLAN OF ABUTMENT

FIGURE:4

For the piles under group-A, vertical load carrying capacity was obtained by primarily considering the end-bearing on rock. Wherever the rock level was above the anticipated scour level, the rock layer was treated as strata not vulnerable to erosion. The piles were socketed inside hard rock by a length equivalent to three times the diameter. It was established that the horizontal load carrying capacity of the rock is adequate with the given embedment of pile. The structural design of pile was made considering the piles as free standing columns, fixed at the pile cap level and at rock level, with the point of contraflexure at centre.

For the piles under group-B, the vertical load carrying capacity was obtained considering end-bearing resistance and also skin friction offered by the sand strata in which the piles were rested. The piles were embedded in soil beyond the maximum scour depth by a minimum length of one-third the length up to scour level. The structural design of pile was made considering the piles as free standing columns, fixed at pile cap level and at the effective level of fixity inside sand, calculated from Reese and Matlock recommendation[3]..

The variations of vertical load in the most lightly loaded and heavily loaded piles are reflected in figures-5 & 6

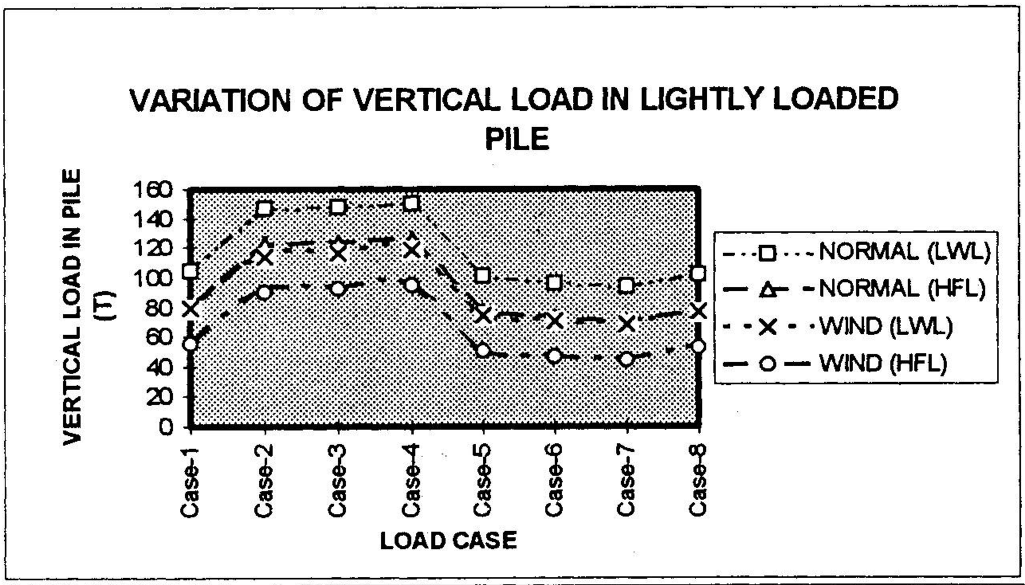


figure-5

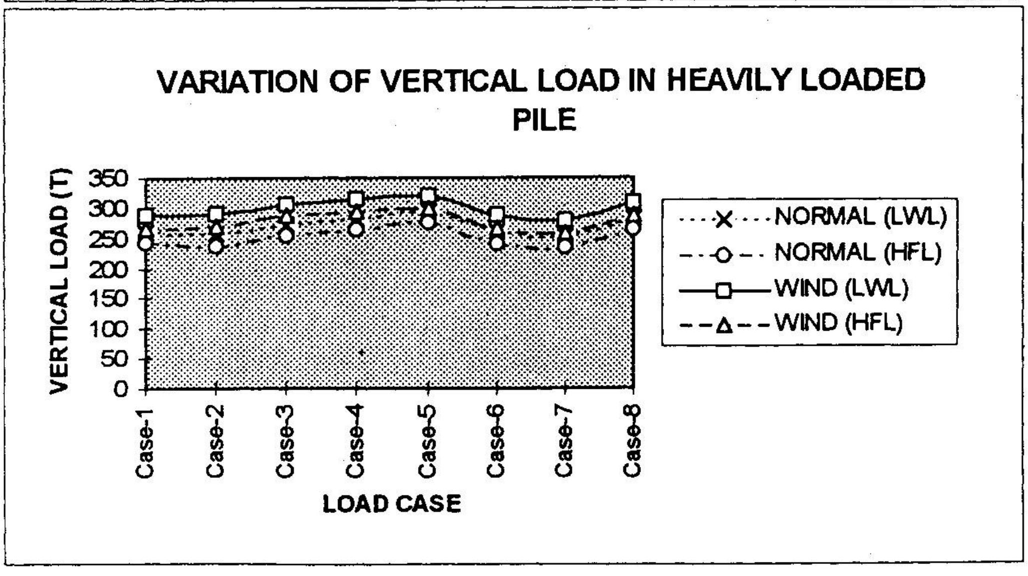


figure-6

8.0 CONSTRUCTION OF FOUNDATION AND SUBSTRUCTURE

During construction it was observed that the low water level (L.W.L.) as observed at site is 1 m higher than what had been reported during investigation and on the basis of which the execution drawings were prepared. It was, therefore, decided to raise the pile cap by 1 m, so that the same can be cast above water with better quality control. This resulted in an increased free standing length of pile at each pier location, requiring redesign of the pile reinforcements for most of the pier locations. The structural design of piles were progressively amended depending on the actual rock levels met at site. This regular interaction between the designers and the construction agency ensured uninterrupted progress at site. No other major problems were encountered in construction of piles and the substructure , thus avoiding the uncertainities met in foundation construction for most bridges in this region.

9.0 CONCLUSION

The foundation selection for this road-cum-rail bridge presented interesting alternatives to the designers. The solutions adopted were highly appropriate and allowed the foundation work to be completed within planned periods and without any major site problem.

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Foundation of Bridges on River Ganges in India

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Summary

River Ganges is a very mighty river of India flowing from the snowclad Himalayan Peaks in the North to the Bay of Bengal in the east. Many important civilisations have grown along its banks, resulting in development of many big cities on them. The river bed is rocky to bouldery in the begining and consist of alluvium sand and clay in most of its reach.

The descharge is quite large and flows in many kilometers in width at many places resulting in big meandering tendencies and large concentration of flow. The scour of the bed is also quite substantial which results in very deep foundation. Dozens of Bridges have been constructed over this river which vary in lengths and have different span lengths and different type of structures. But surprisingly, all of them have one type of foundation, i.e. well foundations.

The paper presents the salient features of these bridges with the details of some important requirements of foundations. This will indicate the suitability, feasibility, versatility and economy of well foundations in very challenging conditions. The paper also deliberates on some of the problems faced during sinking and precautions to be taken. Four case studies of different problems faced during construction are also being presented.

Prayer to Goddess Ganges

भगवति तव तीरे नीरमात्राशनो अहं
विगत विषय तृष्णा: कृष्णमाराधयामि ।
सकल कलुष भङ्गे स्वर्ग सोपान सङ्गे
तरल तर तरङ्गे -देवि गङ्गे प्रसीद ॥

(" O Goddess ! May I worship my lord Shrikrishna on your sacred banks quenching my thirst only with your water, detaching myself from all wordly temptations. O Goddess ! Thoust who undost all my misdeeds and sins, Thoust who is like an access to Heavens .O Goddess Ganges ! Be Happy on me")

The Ganges - The Sacred River

River Ganges is supposed to be the mightiest and holiest rivers of India. Its origin is at Gomukh/ Gangotri in the snow capped Himalayan ranges in District Uttarkashi at Latitude 30.55' N and longitude 79 7' E and after travelling about 2525 Km. along its winding course through the State of U.P., Bihar and West Bengal it falls into Bayof Bengal near Calcutta. Other mighty rivers like Ramganga, Yamuna and Ghaghra meet this river near Kannauj, Allahabad and Chhapra, downstream of Ballia distt. in UP, respectively. The total catchment of this river is 1015275 Sq. Km.
Before Indian Indepehdence or say even upto 1958 there was no independent road bridge over River Ganga right from its source to its tail end. There were, however, only 7 Road cum Rail Bridges at (1) Balawali (2) Garmukteswar (3) Rajghat (4) Kachchllaghat (5) Kanpur (6) Allahabad-Curzan and Izat Bridge and (7) Malviya Bridge at Varanasi. Since then about 13 Bridges have been constructed over this river in UP and 2 in Bihar and 1 in West Bengal and One bridge is under construction in these states.

Foundations - Well Foundations

The salient features of all these Bridges are given here which will indicate the wide range of discharge, velocities, flow condtions with large meandering tendencies, widths and scour conditions, large variation in depth of flow and sub soil conditions as well. It, then, is a revealing fact that the foundations of all these major Bridges built in very challenging and complex situations are all on well foundations.
The variation in discharge is from 10000 Cumecs to 100000 Cumecs , in depth of water from 6 m to 20 m, in length of Bridges from 641 to 5545m, in the depth of scour from 5 m to 36 m below LWL , in dia of well from 5 m to 13.26 m and in depth of foundation from 17 m to 65 m below LWL sunk in bouldery, sandy, clayey and hard stiff clayey stratas.

This foundation system has shown its versality, feasibility and economy practically for all the condtions of river flow and behaviour, resulting in its wide acceptability and use in India

Name of the Bridge	Discharge (Cumecs)	Depth of Flow(m)	Length of Bridge (m)	Dia of Well(m)	Depth of Well(m) (Scour LWL m)
1. Chandighat, Hardwar	19114	10.0	1257.0	6.00	17(8.35)
2. Garmukteswar	9943	16.0	704.5	9.52	33 x4.88 (Dumb shaped well)
3. Farrukhabad	15625	15.0	641.7	5.5	26.9
4. Kannauj	20000	6.0	820.0	5.5	32.21
5. Jajmau,Kanpur	22470	8.0	713.0	5.5	38.35 (22.4)
6. Nanamau	20500	7.0	740.5	5.25	40(26)
7. Gigason, Raebarei			1036.4	5.5	30
8. Phaphamau Allahabad.	27000	11.0	963.4	5.5	29
9. Shastri Bridge Allahabad.	26350	11.0	2083.0	5.5	30
10. Ram Nagar Varanasi	46180	15.1	920.5	13.0	65
11. Mirzapur	42475	-	1001.0	5.5	33
12. Gazipur	50970	12.5	1022.0	8.5	43.5
13. Bhagalpur	94000	11.5	4367.0	11.66	64.70(36)
14. Patna	99600	9.5	5575.0	13.26	55.65

(Photos P1 to P6)

Some Problems faced in the construction of well foundations

These problems may be because of -

1. Short term effects - tilts, shifts and cracking etc. during sinking.
2. Long terms effects - Scouring, non uniform strata, constant overloading, eccentric loading and deterioration etc.

The problems as such arise during construction itself or during service period.
The problems are mainly because of the following factors -

(a) Hydraulic -

(i) Incorrect assesement of design discharge and highest flood level.
(ii) Wrong assesessment of the concentration of flow near the foundations and ignoring its effect.

(iii) Wrong assessment of the scour because of the use of a wrong formula without knowing its limitations.

(iv) Wrong or inadequate assessment of sub soil parameters which also result in wrong assessment of scour.

(v) Incorrect assessment of the velocity of the river or tidal or wave effects.

(vi) Incorrect assessment of the buoyancy forces in case of floating cassion etc.

(b) Geotechnical

Lot of problems, which may arise because of improper and inadequate Sub Soil & Geotechnical investigations, may be because of. -

1. Selecting unsuitable type of foundation.
2. Selecting a weak or improper strata for resting the foundations.
3. Assuming wrong values of soil parameters, which may result in selecting faulty or ineffective technique for construction.
4. Fault or adverse dip in case of rock foundation.
5. Non uniform soil strata at the base or in the grip zone.
6. Artesion Conditions.
7. Seismic effect.
8. Wrong assessment of the soil foundation interaction.
9. Improper methods and techniques of construction.

These create problems of excessive tilting or stucking of well during construction, or excessive tilting or settlement during service . In many cases well foundations tilt because of being seated over a non - uniform strata, part being on rock and part being on soil, or the well resting on slopy strata or uneven strata. Poor or weak strata at the base or poor soil properties in grip zone as compared to those considered in the design, may result in excessive settlement.

The remedy to all such problems is possible only if the tilt and shift and settlement are within permissible limits and have resulted in controllable settlements.

The artesian condition may also result in non workable and unstable conditions and needs to be controlled immediately. This may cause sudden and uncontrolled sinking by own weight of the well and result in excessive sinking, cracking or tilting etc.

<u>Preventive and Remedial Measures.</u>

The problems can be dealt with proper preventive measures or methods and techniques during the construction stage and by proper rehabiliation scheme in service stage.
Some of the measures usually adopted are as follows-

1. In case of sinking in the boulders, stronger cutting edge with an inclined inner plate

and better grade of concrete in well curb be used . The boulders may have to be carefully blasted to remove big boulders and to shake the well.

2. Grabing operations should be regulated so that the presence of big boulders may not obstruct sinking in some part and result in slipping, tilting or shifting of well. In such case excess grabing should be done on higher side. Sometime Divers have to be sent to take out the boulders under the cutting edge.

3. Where the grabing operations are not successful in hard strata, dewatering is done to make open excavation possible.

4. Where dewatering is not possible external loading is done bysome suitable load. Such type of loading is also resorted to where the well is stuck up in some strata . In such cases, loading is further supplemented by water jetting or air jetting etc. also.
5. Use of strutting techniques , may be with use of sal- ballis, can be made to check tilt of the well.
6. Pulling the well withropes etc is also done to correct the tilt.
7. Eccentric loading on the well is also done to rectify the tilt.
8. In case the steining cracks because of some problems in the sinking, which may permit water to seep through and at lower depths shoot out under some pressure, it causes difficulty in doing the repair work. These cracks have to be sealed with neat cement at far as possible. Wider cracks should be plugged with pieces of gunny bags soaked in cement mortar. At still lower levels a method is devised to collect the water and allow it to be discharged inside the well without fouling the space where concrete is to be placed.
The cracked steining is strengthened by laying another RCC steining inside the cracked well, in contact with the old steining.
9. The artesian conditions met during sinking of well of foundation are to be controlled and stabilised by creating a head of water inside after constructing false steining etc.

Case Studies

1. Sinking of a Well foundation. in a depth of 64.70m below LWL for a 4.4 Km. long Ganges Bridge

The 11.6 m dia single circular well foundation was to be sunk to a depth of 64.70 m below LWL. The discharge of the River Ganges at the site being 94000 cumeces, the calculated maximum scour depth being 36 m below LWL, the water depth at HFL being 23 m above bed velocity being 4.5 m/see. The foundation well was started to be sunk when the water depth was about 12 to 13 m.

The soil strata in general was sandy in the upper layers upto 25-30 m below bed followed by very hard and stiff clay layer, sometimes mixed with kankar. However , in the above location, there was hard clay all through with a intervening layer of hard stone in about 2 m depth. The work of this foundation was started in the month of March by making a 13 m heigh sheet pile Island filled with sand . The curb was cast at the top of this island and sunk through this filled up sand. After the well reached the bed of the river it met with hard and stiff clay strata mixed with kankar. The river bed was sloppy towards one side which resulted in tilting of the well. The well had to be loaded eccentrically against the direction of tilt with about 180 T load. The well was also tied by means of wire ropes with the next well which had already been completed earlier and bottom plugged. The sinking was done by chiselling and grabbing. The weight of the chiesel used for this was 2.5 T.

The rate of sinking achieved in this layer was about 1.98 Cm/hr.

The clayey layer was followed by rocky strata at about 18 m below the bed. The well was loaded by concrete blocks of about 180 T and sinking was done by chiselling and grabing operation.

The rate of sinking achieved in this layer was about 1.41 Cm/hr.
The hard and stiff clay layer which followed after this was dealt with by chiselling and grabing as above but under loaded conditions with 250 T concrete block over the well. The dewatering of the well was also done to make sump of the required depth to facilitate sinking. The rate of sinking in this third layer was 1.64 cm/ hr. The well was finally sunk to its foudning level about 65 m . below LWL in the sandy strata . The total crane hours for sinking were 3500 hrs.

(Photos 1 to 4 and Sketch - 1)

2- Revival of Submerged Curb

The location of this foundation was in the mid - stream of river Ganges. The diameter of well was 11.66 m with steining thickness as 3.1 m and curb was of height 5.54 m. The curb was proposed to be cast in two lifts of 3.12 m and 2.42 m respectivelty . Here also, the sheet pile island of 16 m dia was constructed at the location of well in a water depth of about 10m, for laying the well curb at its top. The concreting of first lift involving 180m3 of concrete was done in 2 days.

After about One and half hour of the concreting, leakage of sand from the island occured in the upstream side. As a result , the entire curb, after getting tilled towards upstream side, sunk nearly 4.50 m below the water level.

The tilt was corrected by performing the grabbing in between the curb and sheet pile island in the down stream side. After correcting the tilt as above under water, and making the curb vertical, another sheet pile island was made inside the earlier island at a gap of 0.75m. This gap was filled with clay and bags filled with clay. After filling the gap, the dewatering of island was done by means of 4 submersible pumps of 20 KW capacity to a level lower than the concrete level of curb. The shuttering was then erected and reinforcement fixed and immediately concreting of further steining was done. This operation of dewatering and concreting was continued for nearly 24 hours to complete the concreting of second lift, which also brought the curb top above the water level. The whole process took one complete month. Details shown in the sketch - 2

3. Revival of Over Sunk Well

Some times , the well sink suddenly because of excessive grabing or sump or some weak soil layer and the steining of well , done upto that stage which is always kept above the water level by regulating the sinking and concreting operations , goes below the water level making it difficult to continue further work on steining.

In this case , a well foundation was being sunk in deep waters of River Ganges. The total height of steining except last 2.00 m was completed and the well was left to be sunk finally by about 7.5 m to reach its founding level. As the strata was of stiff clay in which the well was stuck up , chiselling and grabing operations were continued to make a sump of about 8 m below the cutting edge. This position of well remained for quite some time. Then one day the well sunk suddenly by 8.91 m, 1.43 m extra below the final founding level. Due to this extra sinking the top of steining also went below the water level by about 3.50 m. All this happened just before the monsoons when the water level started rising and work could not be continued. After the flood receded and water levels normalised, the work was resumed . It was found that the depthof water was 2.5 m. The well was in side the river bed and about 25m from bank. Sand embankment was made from the bank to extend and cover up the location of the well. A cutting edge of 2 m more dia than the dia of well was placed there and a cofferdam with 30 Cm steining thickness was made and got sunk to nearly 0.50 m below the top of the steining of the over sunk well. The concrete cofferdam was then dewatered and steining of the original well was exposed making it feasible to build up further steining to the required level. Details shown in sketch- 3.

4 - Scouring of Foundations during Construction Stage - Programme for Reconstruction

Two well foundations of a Bridge under construction over River Ganges were scoured below the cutting edge level during the floods and were carried with the flow and laid almost horizontally. The location of the well could not be traced as it was buried inside the scoured bed , later on silted A number of methods were tried to locate the top of these well. When wells could neither be seen by naked eye nor could be located by various methods , i.e., weight dropping method. Echo sounding method and through divers after the water level receded,exploratory borings had to be done to trace out the position of the well foundations. This is shown in the sketch.

From the perusal of the data it was seen that at the time of floods these two wells were already sunk to a safer level as far as the normal design scour was concerned and the other wells sunk to this level the same time and in previous years stood well during the floods. Moreover, the design HFL was also not achieved (was about 1.5 m less) which [illegible] margin of safety in the depth of foundation at that time. The [illegible] revealed that there was probably an unprecedented [illegible] of the Bridge which resulted in excessive [illegible] and [illegible] wells

Proposal for Reconstruction.

As the scoured wells were still obstructing the old foundation locations, the span arrangement had to be changed and two proposal were considered for laying the new well foundations.

Alternative-1

By laying the new well foundation on island to be constructed with steel sheet piles.

Alternative - 2

By floating Steel Caission and Gantry Arrangement

Because of thechange in flow conditions and excessive scouring in this zone, the depth of water was in the range of 17m to 22 m. For such deep depth of water alternative I which was the construction Technique adpoted for other wells, was not feasible.

As such alternative II is to be used. This is the arrangement of fabricating steel curb and steining segment, lifting in vertical direction, shifting and placing the same on the barge and taking it to the location of the well foundation, lifting with steel gantry arrangements on the gantry barge, removing the caisson barge, lowering the steel caissons segment with the gantry arrangement and concreting of the same in such a way that it remains floating all the time due to its bouyancy effect till it touches the bed level of the river at the foundation location, sinking a little bit more as considered safe for laying another segment of steel caisson over it so that the bottom of steel caisson rests in the normal bed of the river and top of the last steel caisson segment remains above the water level such that the balanced steining of the caisson could be done directly by fixing shuttering.

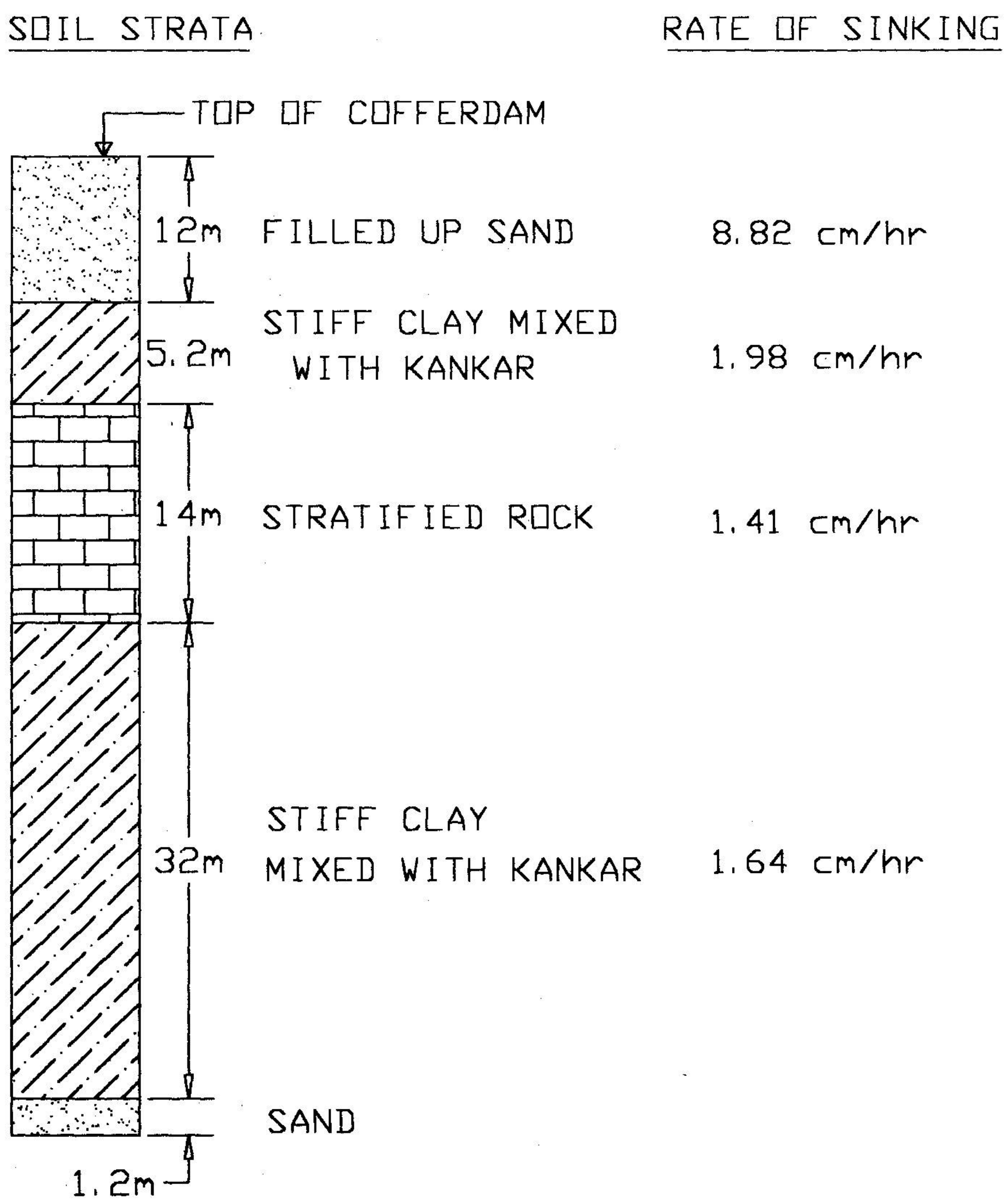
(Sketch 4 to 5)

The above method has already been tried elsewhere and has been quite effective and useful.

Conclusions

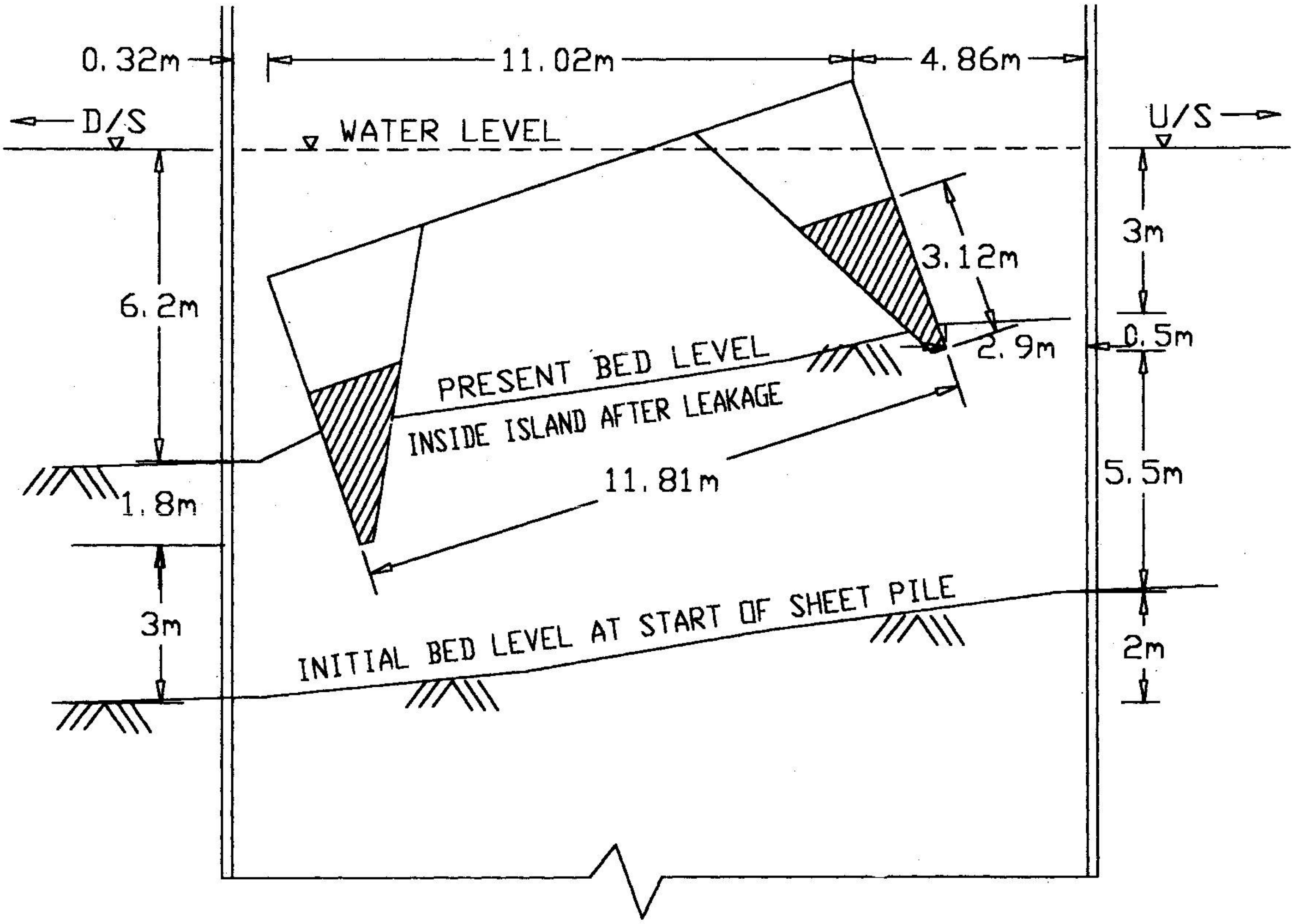
Use of well foundation for so many Major Bridges on such mighty river in different conditions indicate that these type of foundations are the best solution for the scouring rivers.

The case histories as presented above have however, been discussed here to bring out a word of caution to the design Engineers to have proper and adequate hydraulic and Geotechnical Studies and investigations done and to construction Engineer working in the bed of such mighty rivers having very large depths of water and discharge, to remain alert to the flow condition of river and scouring pattern so as to take timely preventive measure to check such happenings which take lot of time, effort and money in ractification.



CASE STUDY 1 - DETAILS OF STRATA & RATE OF SINKING

SKETCH 1



CASE STUDY 2 - REVIVAL OF SUBMERGED CURB

SKETCH 2

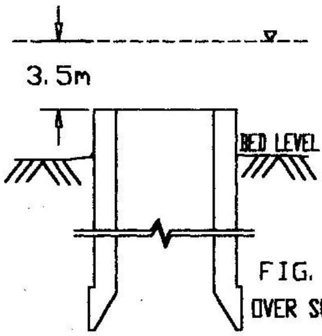


FIG. 1
OVER SUNK WELL

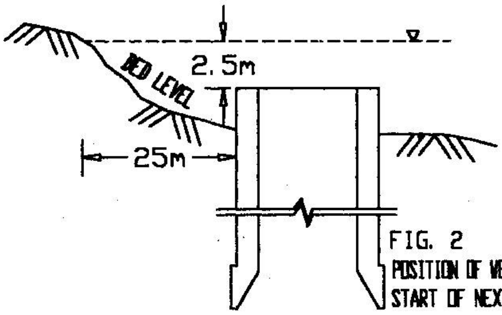


FIG. 2
POSITION OF WELL AT
START OF NEXT SEASON

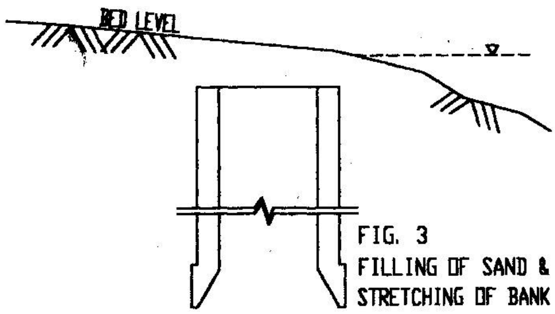


FIG. 3
FILLING OF SAND &
STRETCHING OF BANK

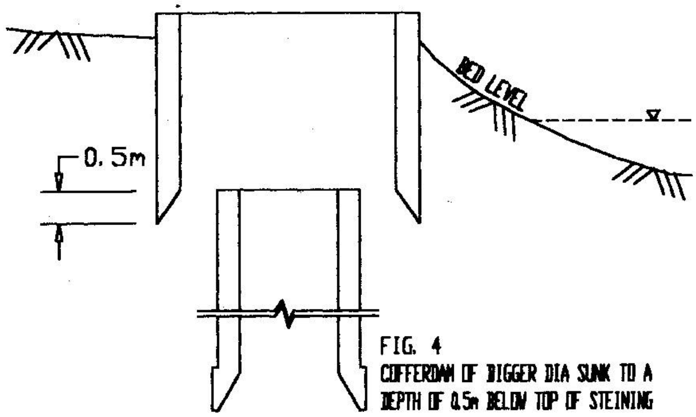
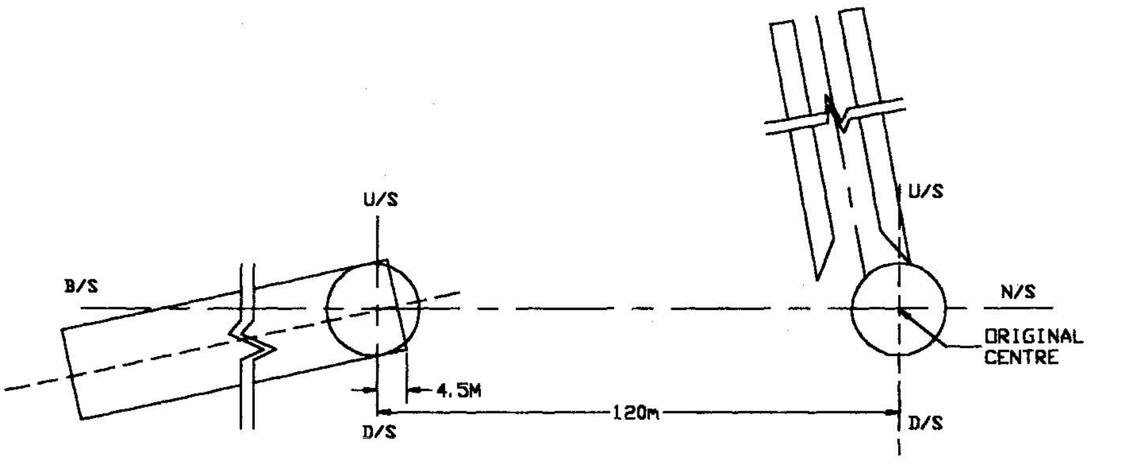


FIG. 4
COFFERDAM OF BIGGER DIA SUNK TO A
DEPTH OF 0.5m BELOW TOP OF STEINING

CASE STUDY 3 - REVIVAL OF OVERSUNK WELL

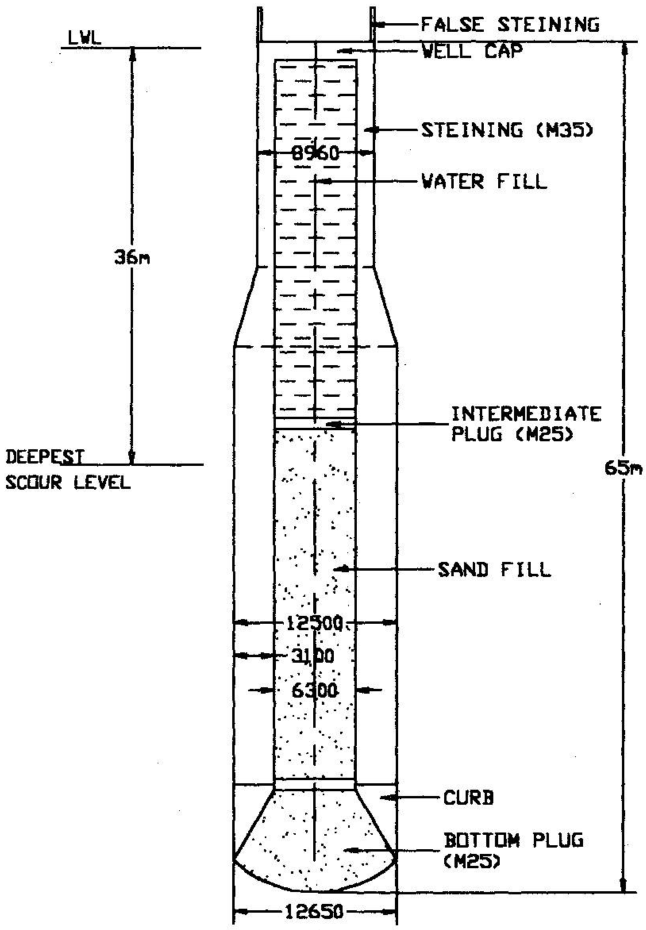
SKETCH 3



POSITION OF WELL AS ASCERTAINED BY BORING DONE AT & AROUND WELL

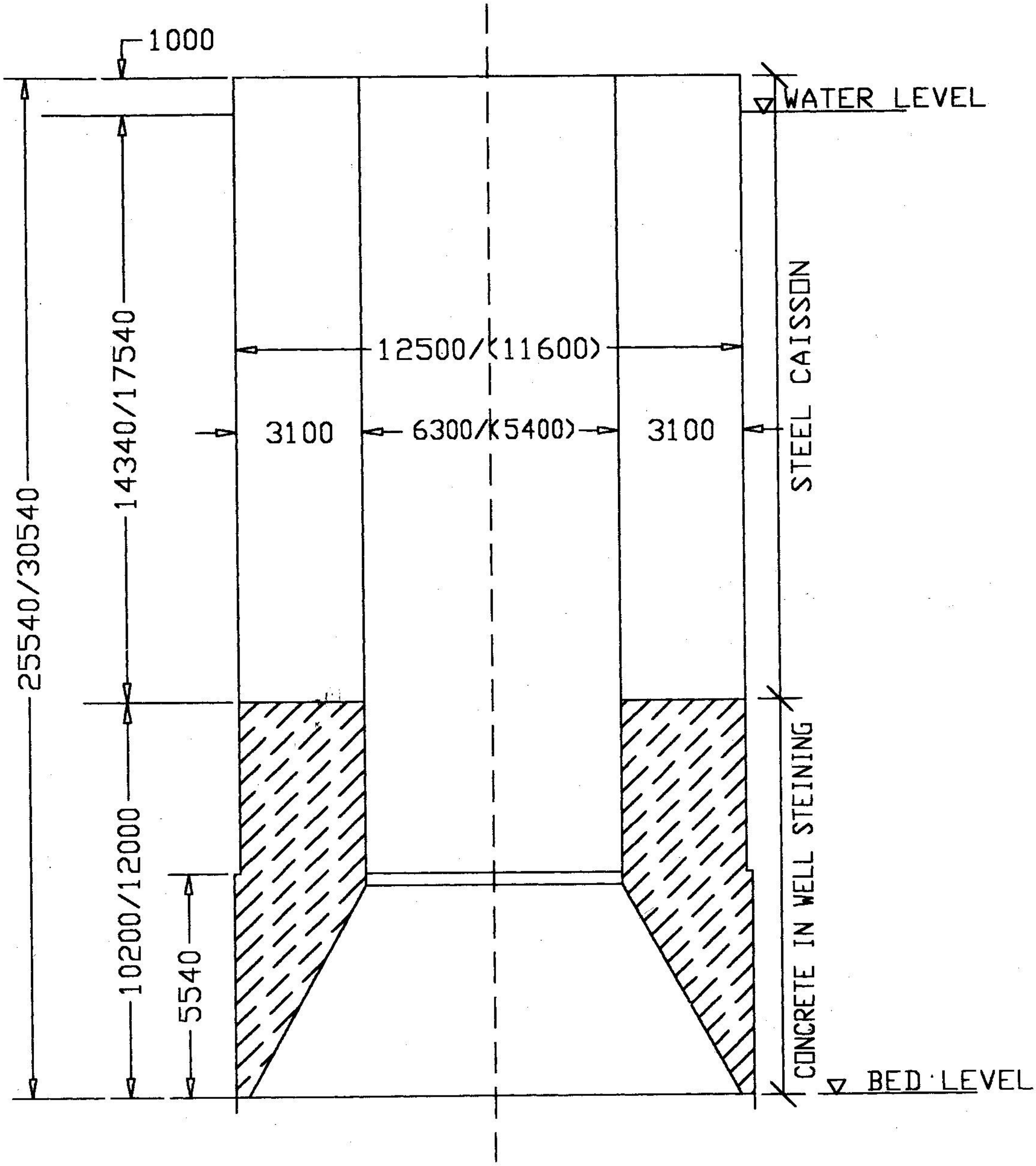
POSITION OF WELL AS ASCERTAINED BY BORING DONE AT & AROUND WELL

CASE STUDY 4 - SCOURING OF FOUNDATIONS DURING CONSTRUCTION



CASE STUDY 4 - TYPICAL CROSS SECTION OF PIER WELL

SKETCH 4



CASE STUDY 4 - LAST STAGE DURING GROUNDING OF CAISSON

SKETCH 5