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Case Histories of Retaining Structures and Deep Excavations. Paper No. GR-V

Raj V. Siddharthan
University of Nevada, Reno, Nevada

Arni E. Rafnsson
Honnun Ltd., Reykjavik, Iceland

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Case Histories of Retaining Structures and Deep Excavations

Raj V. Siddharthan
Professor of Civil Engineering, University Nevada, Reno, Nevada, USA

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INTRODUCTION

Design of retaining structures and deep excavations have been evolving over the years. In the recent years the construction of these facilities have been in highly problematic soils, on which construction in the past have been avoided. Engineers are beginning to understand better the behavior of retaining structures under variety of complex loading situations. Many readily available computational tools are successfully being used to predict the retaining wall and deep foundation behavior. The computational tools are very powerful because when used correctly they can point to any disaster prior to the construction. Through parametric studies they can also help to arrive at an efficient design. Many papers which are summarized below used numerical methods to predict behavior and subsequently have attempted to verify the applicability of the numerical methods used. Detailed study and interpretation of case histories is a necessary and extremely important step in the verification of the theories used in the numerical models.

The 14 papers submitted to this session have been summarized below.

SUMMARY OF PAPERS

Paper No. 5.01: "Failure of Excavation Bracing System for New World Plaza at Changchun in China," By M.S. Wang, X. Wu, and M.C. Wang.

A case history on the failure of an excavation bracing system used in the construction of the New World Plaza in Changchun in China is described in the paper. The New World Plaza consists of a 3 storey basement and a 8 storey main building, which in turn supports a 42 storey tower. The tower portion of the building was founded on piles while the rest of the building was founded on a mat foundation. To facilitate this design, a 15 to 16 m deep excavation was made over a 105 by 80 m area. A total of 19 borings with six of them extending to a depth of 40 to 50 m deep were made to determine the subsurface condition. The site is generally a miscellaneous fill in the upper 11 m underlain by 13 m of alternating clay and silty clay layers and below which coarse sand and weathered shale extended to greater depths. The soils had a unit weight around 20 kN/m^3; cohesion about 50 kPa for the silty clay and about 120 kPa for the clay layers. The coarse sand had a and friction angle of 35°. Ground water table was observed at 2.4 to ~ 4.0 m below the ground surface.

The bracing for the excavation consists of 157 anchored cast-in-place reinforced concrete piles 1.0 m in diameter, 19.5 m in length with 2.2 to 2.6 m center to center spacing. The piles were tied back with two anchors on the wales at the midway between two adjacent piles. The wales were placed at 5.0 and 10.0 m below the top of the pile. In total there were as many as 312 anchors. The anchors were installed in a 150 mm diameter hole dipping at 15° to 22° downwards into the ground. Anchored length was 13.5 to 14.5 m and unanchored length was 3.0 to 5.0 m. The design anchor pullout strength was 25 MPa . The anchors were prestressed to 310 to 350 kN. The ground water table was lowered using 28 wells spaced at 10 to 15 m. These wells were 0.6 m in dia and penetrated to about 27 m into the ground. In addition to these wells, which were located along the periphery of the excavation, there were four more wells within the excavation area.

When the excavation reached the design depth of 16 m, a wale located on east side of the excavation broke and subsequently 8 piles in the vicinity also collapsed. The west side of the excavation also showed considerable distress and subsequently 2 months later 7 more piles on the west side collapsed.

An investigation of the failure was undertaken and the investigation team attributed the failure to the following:

(1) Improper installation of the tieback anchors. The anchors were not at the design spacing of 5 m. Some of the anchors were not at the center between the piles.

(2) Inadequate stiffness and strength of the wales. Old rails were used as wale without any consideration given to the reduced fatigue strength of the rails.

(3) Adverse effect of freezing temperature. Mean lowest temperature was -25° C and the freezing period lasted for 4 months. Silty clay layer, which was exposed due to the excavation underwent swelling resulting in bulging of soil at many locations.

Solution to the problem was arrived at after a further site investigation. It proposed reduction in the area of excavation, and reduced mat foundation size. Instead a portion of the building was founded on 8 m length piles.

This case history is a good example relative to problems arising from substandard engineering construction and design.

Paper No. 5.02: "An Anchored Band Bracing System for Deep Excavations in Clay," By Linbing Wang.

This paper describes the design and the use of an anchored band bracing system to support a large excavation. The method was introduced for the bracing of a deep excavation for the underground transportation system of Hefei Electrical Power Plant located in Hefei, China. The excavation was approximately 168 m long, 32 m wide, and 12 to 14 m deep (Fig. 2 in the paper). The excavation was in hard clay with low swelling properties. The excavated area was for coal storage and the bracing system on the slope around the area was required to stabilize the slopes of the cut. Where the train track is closest to the cut it is only 1.25 m from the edge of the slope.

The following options to stabilize the cut were considered: steel sheet piling, pre-drilled soldier piles and anchored band bracing. The last options was chosen, primarily for its cost effectiveness. The anchored band bracing consists of pre-stressed anchors, two reinforced concrete bands all the way around the excavation and shotcrete surfacing. The spacing of the anchors was 3 to 4 m horizontally with the closer spacing at the end parts. The Anchors were 14 to 16 m long with design prestress of 150 kN.

The anchored band system reinforces the slope through two important mechanisms: increase pressure on the failure surface, and additional anchor shear resistance. The author computed the increase in normal stress along the failure surface from anchor pre-stress using simple elastic half-space solutions. The shear resistance by the anchors was estimated by the "rule of mixture" as follows: $\tau_{sa} = \tau_s(1-\eta_a) + \tau_a\eta_a$ where τ_{sa} is the shear strength of the soil-anchor composite, τ_s is shear strength of the soil, τ_a is shear strength of the anchor and η_a is the area fraction of the anchors. The design of the anchors was verified by pull out test of two anchors. The bracing system performance was as expected and no problems related to the bracing were evident during the construction.

Though the water table was well below the excavation, there was evidence of seepage from rainfall at the surface during the time of construction. No special remedial measures were taken. However, subsequently after 5 months after the excavation, a five-day rainfall caused a partial collapse of a 6 m section of the excavation. The collapse was shallow (about 2-3 m deep) of length 18 m and it was quickly identified and repaired.

The proposed anchor band system has been a cost-effective design. If the construction had been planned to avoid the rainy season this bracing system would have worked as designed.

Paper No. 5.04: "Prediction and Performances of Short Embedded Cast In-Situ Diaphragm Wall for Deep Excavation in Bangkok Subsoil," By Wanchai Teparaksa, Narong Thasnanipan, Aung Win Maung, and Wei Shixin.

The paper presents a case history of performance of a diaphragm wall which was subjected to loads other than the original designed loads because of subsequent modification in design. The project is located in the vicinity of the main central business area of Bangkok. The structure is a multi-story business complex with a basement. The load of the structure was carried by bored cast-in-situ piles and barrette piles with length extending to depth of 60 m below the ground surface. The subsoil conditions at the site was generally a 15 m thick soft to medium clay with an undrained strength of 1-3.5 t/m^2, underlain by a stiff clay with an undrained strength of 13.7 -18.0 t/m^2, extending to a depth of about 22 m. Below this depth a dense sand with SPT blow count varying between 22 and 37 was encountered. The constructed cast in-situ diaphragm wall is 0.8 m thick embedded into the stiff clay extending 18 m below the ground surface. The maximum excavation depth was set to be 14 m and therefore the embedded depth of the wall was set at 4 m in the original design. A three level of temporary bracing system was proposed to construct the basements with conventional bottom up construction method. Finite element (WALLAP and CRISP) and finite difference (FLAC) based computer programs were used to verify the original design. These programs simulated the staged excavation and predicted the wall deformation behavior.

After the construction of the diaphragm wall it was decided to increase the number of stories of the basement by one. This reduced the embedded wall depth to 2.2 m. Another computer model analysis was carried out for the new condition. The analysis indicated that a wall movement at the toe will be around of 23 mm, while the movement at the top was estimated at 77 mm. Based on the computer modeling the following recommendations were made: (1) monitor the diaphragm wall movements during further excavation; (2) construct 20 cm thick 6 m wide reinforced lean concrete slab in front of the wall when the excavation depth reached 13.8 m; and (3) excavation to 15.8 m should be carried out leaving a 12 m wide soil berm with the concrete slab on top of the berm all around the wall. During construction 4 inclinometers and 16 settlement observation stations were installed. The bracing system included continuous waling beams along the diaphragm wall and longitudinal and transverse struts. The spacing of the struts was about 6.6 m.

Observed wall movements were 55 - 98 mm at the top of the wall and 1 - 16 mm at the bottom compared to the predicted values of 77 mm and 23 mm, respectively by the computer program. The maximum observed ground settlements behind the wall were generally 34 - 50 % of the measured lateral wall movements. In one area the ground settlements matched with lateral wall movements. The wall performance was overall satisfactory.

Paper No. 5.06: "Excavation Behavior of Soft Marine Clay Deposit in South Korea," By Young-Nam Lee, and Seung-Won Lee.

Construction of a sewage treatment plant having the site area of 160 m by 520 m and constructed on a soft marine clay deposit of average thickness 15 m is discussed in the paper. The subsoil at the site is a clay of low plasticity with natural water content about 45 %, plastic limit 15 to 22 %, and liquid limit 35 to 45 %. The undrained shear strength, determined by field vane tests ranged from 15 to 35 kPa. To construct pumping station and sedimentation basin of the plant, an excavation of 59.3 m by 31.5 m in plan to a depth of 10.5 m using sheet pile wall system with struts and anchors. Furthermore, a jet grout-pile foundation system was adopted to support superstructures and to protect basal heave failure of the clay while the excavation proceeded. The jet grout-piles were of 1 m in dia and were spaced at 2.1 m. The clay gained strength slowly with curing time of grouting. Shear strength measurements showed that after 50 days of curing the shear strength at all measured locations was more than the original strength.

The excavation was monitored with using 3 inclinometers, 4 load cells, 3 strain gauges, and piezometers. The struts were located at 1.5 and 4 m from the surface. The paper provides data on the horizontal movements shown by an inclinometer as the excavation proceeded. The maximum horizontal displacement of 7.6 cm was recorded at a depth of 6 m when the excavation depth reached 7.5 m from the surface. Although it is not clearly pointed out in the paper it appears that the construction was successful though significant horizontal displacements were observed on the sheet pile wall.

Paper No. 5.08: "Shoring Analysis, Design and Construction at the Seattle Symphony's Benaroya Hall," By Carole L.B. Mitchell, and Monique A. Nykamp.

The construction of the Benaroya Hall, the new home to the Seattle Symphony downtown Seattle, introduced interesting construction challenges. When completed, the building will have an approximate area of 187,000 square feet, and the main auditorium will have a seating capacity of 2,500. The site is bounded by 2^{nd} and 3^{rd} Avenues and University and Union Streets. The surrounding buildings had to be maintained and protected from the excavation. This concert hall also borders a twin transit tunnels and the underground University Street Station of the Seattle Metro Transit (Metro). Since the construction of concert hall removed soil within 25 feet of the transit tunnels and the station, many serious concerns existed regarding potential movements of these structures. A combined system of soldier pile, tieback walls and soil nail wall was used to stabilize the excavation that ranged from 15 to 50 ft deep. The paper describes in detail many aspects of engineering concerns, especially relative to excavation near the Metro station. The study includes numerical prediction before construction, monitoring during construction and subsequent interpretation of the measured data as the construction proceeded.

Soil borings indicated variable subsurface soil condition, very dense clean to silty sand with varying percentage of gravel and hard clayey silt to silty clay overlying glacial till or till-like soils. Generally SPT resistance values were greater than 100 blows per foot. From observations made during drilling and from measurements taken in observation wells installed in two borings, it appeared that perched groundwater existed at the site. However, during construction only limited amount of seepage was observed.

A finite difference based analysis (FLAC) under plane strain conditions was carried out to analyze the behavior of the soil-nail shotcrete wall, which was to be constructed adjacent to the Metro station (3^{rd} Avenue). The program can also accommodate beams and cables elements so that walls and soil nails along with soil elements can be represented in the numerical model. Horizontal and vertical spacing used in the analysis was 5 ft. The program FLAC predicted a maximum soil movement in the order of 0.3 in. Numerical studies also included the computer program SNAIL (version 2.11), which was developed by the California Department of Transportation (CALTRANS). This program computes a factor of safety against slope failure for a given soil nail spacing, length, and load transfer capacity.

Performance of the soil nail wall was monitored through measurements of an inclinometer installed between the Metro station and the shoring wall face, and through weekly optical surveys on nine angle irons attached to the top of the wall. According to optical surveys, the angle irons moved between 0.0 and 0.36 in at the maximum excavation depth. The inclinometer measured a maximum soil movement of 0.1 in. When compared with the predictions given by FLAC, the measured displacements were smaller.

The north, south, and west sides of the concert hall site were shored using soldier piles and tiebacks. Up to three levels of tiebacks were used. The wall height varied from 15 to 50 ft. For tieback design, an allowable skin friction value of 1 ksf was used to size the anchors. The wall design was based on a trapezoidal pressure distribution with maximum pressure equivalent to 20H, where H is the height of the excavation. Monitoring during construction indicated that range of deflections, at maximum excavation depth, was on the average about 0.2 in with a maximum movement of 0.5 in for the shoulder pile.

Paper No. 5.09: "Behavior of Double-row Pile Retaining Structure for Deep Excavation in Soft Clay," By Cai yuanqiang, Wu shiming, Chen yunmin, and Xu changjie.

This paper describes the application of an atypical bracing system, consisting of a double-row pile. This retaining structure was used to excavate a depth of 10.5 m in 110 m by 70 m area in a soft clay. The soil at the site consisted of 3.0 m of fill material followed by soft silty clay to clayey silt extending to a depth of 27 m below the surface. The double-row pile system consisted of 0.8 m in dia and 20 m in length piles with a spacing of 4 m between the rows. The first pile row located closer to the excavation was spaced at 1 m, while the second row was spaced at 2 m. Both the rows of piles were connected by beams at the top of the piles at a depth of 2 m from the surface. To reduce the deflection of the pile system, grid-shaped soil-cement columns were constructed in front of the first row of piles and also in between the pile rows. This soil-cement columns provided a substantial passive resistance.

A finite element based analysis was carried out for the prediction of lateral deflection and stress. Measured values agreed relatively well with the analysis. The main advantage of using double row piles was that no bracing was needed, which in turn resulted in a substantial shorter time of construction.

Paper No. 5.10: "Soil Nailing: The MoDOT Experience," By Kevin W. McLain.

Three case histories relative to the construction of three soil nail walls are presented in the paper. Two of the walls were temporary while one was permanent. The paper illustrates in sufficient detail, the initial design assumptions and the design based on these assumptions, and subsequently how the design has to be modified based on field pull-out test results. The initial design specified 150 nails with the embedment length of 18 ft installed at 15°. The nail spacing was at 4 ft horizontally and vertically. The excavation covered an area of 20 ft in height and 150 ft in length. The nails were 1¼ in in diameter with planned minimum grout hole diameter of 6 in.

The in-situ soil at the site consisted of lean clay with liquid limit varying from 30 - 39 and plastic index varying from 9 - 20. Internal angle of friction and cohesion were estimated to be 29° and 300 psf, respectively. The Caltrans developed computer program SNAIL with a factor of safety of 1.5 was used to arrive at the design.

Initial construction of the soil nail wall started before any field verification of the soil-grout adhesion strength was undertaken. An extensive field verification revealed that the soil-grout bond strength has been conservatively assumed at 7.5 psi in the initial design. The field verification consisted of as many as 8 pull-out tests with varying bonded length (12 to 26 ft). The field study results led to changes in design and subsequently the nails installed were 32 ft in length in 8 in holes.

The construction at all sites were completed successfully although some problems were involved. Overall the paper contains many important material to alert engineers about problems that can arise when constructing soil nail walls.

Paper No. 5.11: "Behavior of Braced Deep Excavation in Soft Soil," By Xu changjie, Cai yuanqiang, and Wu shiming

A successful case study of braced deep excavation in Hangzhou in 1994 is presented. Some details behind the design and construction procedure are also described. The project consisted of construction of a 10-story high rise building with a two-level basement. The site boarder two main roads and a high rise building. The excavation pit was 145 m in perimeter and to a depth of 7.4 m. The soil at the site consisted of a surface fill material followed by soft silty clay to clayey silt extending to greater depth. The thickness of the layers were not presented in the paper.

The design consisted of braced cast-in place pile system. Piles were 0.8 m in dia and were at 1 m spacing, extending to a depth of 16 m. Only one set of braces were used and they were placed at 2.1 m from the pile top. In addition, two rows of 0.7 m dia cement mix column were installed behind the piles, which acted as an effective waterproof screen. The bottom of the pit (depth 7.8 m) was strengthened by cement mixed grouting. The thickness of the grout depth was 5 m.

The authors also undertook a preliminary analytical study using the Finite Element Method. This treated pile, brace, waling and the top beam as beam elements. The grouted soil area was treated as elastic. The computed response in terms of pile deflection, and bending moment are provided. Some parametric studies to understand the influence of the stiffnesses of pile and brace were undertaken to arrive at the optimum pile and brace stiffnesses.

The site activities also involved five inclinometers up to a depth of 20 m, and two strain gauges. Data on soil deflection with time at two locations as the construction proceeded are given in the paper.

Paper No. 5.12: "Large Retaining Structures Reinforced by Geosynthetics- French Case Histories," By Jean-Claude Blivet, and Jean Pierre Gourc

Design, construction, and instrumentation of three large reinforced structures supported by geosynthetics have been described. These structures are representative of the type of walls built in France today. The facing of many original geosynthetic-reinforced walls were uncovered and it led to the deterioration of the faces from UV solar radiation. The three cases described in the paper used different facings. They include vertical prefabricated concrete outer wall, slope facing with vegetation, and segmental wall made of concrete units linked to the reinforcing geotextiles.

The first wall with vertical prefabricated facing consisted of three blocks or segments of height 7, 6.1, and 5.6 m, giving a total height for the wall as 18.7 m. This site was instrumented using 3 vertical inclinometers, 13 strain gauges on geosynthetic sheets, and 4 rod-type horizontal extensometers. The extensometers connected the front of the wall facing with rear of the embankment. The extensometer readings were used as a calibration to inclinometer readings.

The second wall had a flexible facing with natural vegetation in soil retained by geogrids. The total height of the wall is 27 m. The instrumentation program included 12 strain gauges, three horizontal extensometers and one vertical inclinometer. The third wall is segmental wall constructed with concrete face units that clamped the geotextile sheets. Direct contact between the geotextile sheets and the concrete was avoided by providing thin sand layers. This precaution was deemed necessary to avoid hydrolysis of the woven geotextile. The instrumentation in this case included 4 horizontal inclinometers and four sensors to measure elongation of the geotextile sheets.

The paper provides the inclinometer and extensometer readings during the construction period and in some cases long term performance up to 3 years of service.

Paper No. 5.13: "Geotechnical and Geological Features of U.S. 189 in Provo Canyon, Utah," By Thomas S. Lee, and Steven H. Brandon

A multi-million dollar, 15-mile long U.S. Highway 189 project through Provo Canyon, Utah is being designed and constructed in phases. Construction to widen and straighten a 2.5 mile long section of U.S. 189 known as the "The Narrows" commenced in December 1995. This road crossed of two large active landslides zones. This project consists of twin 300-foot-long two lane tunnels, 3/4 million cubic yards of soil and rock excavation, 60,000 square feet of cast-in-place concrete soil nail walls, and 90,790 square feet of mechanically stabilized embankment.

Many slope failures and ground deformations are attributed to unique geologic and hydrogeomorphic formations of soil/rock that sometimes underestimated by designers, field staff, and contractors. This paper identifies possible causes that triggered slope failures and tunnel cracking.

During excavation of some of the cuts, landslides occurred that required remediation. Cracks were noticed near the northern portal of the tunnels which necessitated immediate stabilization. Observations during construction are presented.

Immediately north of "The Narrows" section of U.S. 189 is an approximate six-mile segment called the Upper Provo Canyon project. The project includes a one-mile section of roadway that traverses over some landslides, known as the Hoover Slides, which have been active for at least 60 years. The Hoover slides are within a thrust fault known as the Deer Creek thrust. From the exploration program, geotechnical and geologic features were identified which permitted the development of probable chronological events of the Hoover Slides and postulated sliding mechanisms responsible for the movements.

Paper No. 5.15: "Passive Earth Pressure Tests on An Integral Bridge Abutment," By Theodore A. Thompson Jr., and Alan J. Lutenegger

This paper describes a field study to investigate the development of passive earth pressure and the influence of wingwall orientation on full size bridge abutment. A reinforced concrete main retaining wall measuring 15 ft in length by 8 ft in height with a thickness of 18 in was constructed to model the center section of an abutment. The wall was supported on 3 ft wide spread footing embedded to 2 ft into a bed of compacted granular fill. The wingwalls were 6 ft in length 8 ft high with a thickness of 18 in. The wingwalls were left unattached to the main abutment wall so that the geometry of the abutment-wingwall system can be varied. In the Test Series 1 the wingwalls were attached to the main wall with the use of steel bars and tubes.

The testing program involved the application of a lateral load at the top of the main wall using a stiff reaction wall built next to the retaining wall. The testing program was instrumented using 17 hydraulic earth pressure cells with vibrating wire transducers of which 14 were located on the main wall and three on one of the wingwalls. Wall deflection was monitored by two inclinometers cast into the main wall and also by four tiltmeters located on both the main wall and a wing wall. A load cell was used to measure the applied load at the top.

A typical test series consisted of data obtained during backfilling, initial passive loading, and one passive reloading of the wall. In the initial test phase the wingwalls were parallel to the main wall (i.e. wingwall angle = 0°) and in subsequent test phases, the wingwall angle was changed to 45° and 90°. For each test phase earth pressures, wall deflections and applied loads were monitored. The tests were conducted by incrementally displacing the top of the main wall a distance of 0.25 in and were continued until a total wall displacement of 2 in was achieved.

The test results indicate that the passive pressure distribution behind the wall was not as given by classical design procedure. The maximum passive pressure typically occurs at a distance 1/3 way from the top and at the bottom of the wall it reduces to zero. The results also showed that wingwall orientation had a significant influence on the magnitude and distribution of the passive earth pressure.

Paper No. 5.17: "Abnormal Wall Movements of a Steel Silo Caused by Paddy Rice Storage," By Minh Phong Luong

This paper gives a brief overview of the type of past failures relative to large metal silos and presents one case history where a silo that stored paddy failed. This type of structures generates unusual class of both structural and functional problems. The main input to the silo design are the stresses and strains on the silo walls. This requires a realistic constitutive models for the mechanical and also the flow behavior of material being stored. Though more sophisticated and complex models involving viscoplastic characterization has been attempted in the past, the paper uses conventional triaxial tests to obtain the stress-strain relationship for paddy. A set of triaxial test results on paddy in terms of deviatoric stress and volumetric strain as a function of axial strain is included. The paper also provides some practical suggestions for improved silo design with bracing.

Paper No. 5.18: "Numerical Modeling of a Crib-Wall Failure," By Wolfang H.Roth, and Alexander Delnik

A crib wall of 45 ft in height was to support a fill at the Hebrew Union College in Los Angeles. When the wall collapsed less than 2 years after the completion, a number of independent investigations were launched to determine the probable causes of failure. The wall has exhibited signs of distress from the very beginning and in Fall 1990, shortly after reaching its full height, cracking/crushing of the pre-cast concrete crib-wall elements had been observed near the base of the wall. As a result, its tallest section was lowered to 35 ft with a sloped backfill. In spite of this modifications, however, cracking of crib-wall elements continued and on March 1992, a 40-feet long section of the lower wall collapsed.

Following the failure of the wall, a through failure investigation was launched to identify the causes of failure. The paper describes in great detail many aspects of this investigation. The field and other preliminary investigation pointed to the following possible reasons for the failure: (1) Higher strength for backfill used in the original design (2) Structural defects of crib-wall elements, especially the compressive strength of concrete used in crib-wall construction, (3) Higher stiffness of crib-wall foundation contributing to at-rest condition than the active condition, (4) Poor drainage of the wall backfill leading to build-up of porewater pressure and thus larger lateral wall pressures. The paper describes many elements of a numerical analysis undertaken to address the above reasons and to arrive at the real reason(s) behind the wall failure.

The 2-D finite-difference program FLAC was utilized in the numerical investigation. In analyzing the construction stages and subsequent wall failure, a simple Mohr-Coulomb constitutive model was coupled with seepage analysis to simulate the effect of infiltration. Two distinct case histories are presented. The first case involved wall construction through completion, when localized crushing of crib-wall elements occurred even before the rainy season. The second case simulated a prolonged period of rainfall during which the wall collapsed. It was found that the low permeability backfill material allowed the buildup of porewater pressures which triggered the collapse of the wall. However, it was also concluded that neither the initial cracking/crushing nor the subsequent wall collapse would not have occurred had the stacked crib-wall elements possessed the concrete compressive strength specified in the design.

Paper No. 5.19L: "Pier Failure of Bridge and Geotechnical Investigation: A Case Study," By J.K. Jain, A.R. Sexena, and S.K. Shrivastava

This paper describes a case of bridge failure initiated by the failure of the bridge pier. The bridge structure is a simple two span bridge of length 30.5 m. The bridge pier is a coarse rubble stone masonry wall. The base dimensions of the pier are 3.8 m in the traffic direction and 10.1 m in the perpendicular direction. Only very preliminary site investigation (test pits) were undertaken when the bridge was built in 1980-81. During the flood of 1995, the pier foundation unevenly settled causing collapse of the pier. This in turn lead to the deck to fall onto the ground below. The fallen deck was undamaged. The cause of the failure was scouring of the pier foundation due to high floods. By using the same bridge deck the bridge was restored in January 1997.