THE FAILURE AND RIGHTING OF THE TRANSCONA GRAIN ELEVATOR

The foundation failure and righting of the Transcona Grain Elevator in 1913 is a truly remarkable case history made famous by its collapse during filling after foundation pressures exceeded the bearing capacity of the underlying clay foundation soil. The following photographs and documentation provides a record of the bearing capacity failure, how the original designers struggled to understand the cause and most remarkably, how ingenuity and determination led to the successful righting of the 20,000 ton structure. It provides an account of the landmark work carried out in the 1950's ending with a modern perspective made possible by finite element modelling techniques.

This historical account would not have been possible without the enthusiastic support of Mr. Bill Parrish Sr. who kindly provided permission to use the original construction photographs and access to the original elevator, which is now owned and operated by Parrish and Heimbecker. The photographs were originally presented in the Heritage Room at the Canadian Geotechnical Society Conference in Winnipeg in 2003. The interest shown at that time by the delegates was instrumental in the Society's determination to preserve such valuable information from our past.

Two files are provided. The first file contains a photographic record of the failure and righting. The photographs carry short explanatory captions. Because of the large file size, it is recommended that this document be downloaded first before viewing. The second file contains selected papers or articles which may be difficult to locate and a list of references that may be of assistance to those wishing to research this case study further. The file has been bookmarked for convenient access - simply 'click' the bookmark symbol at the upper left of the screen.

Manitoba Fre

'AST. Sun rises, 6:57 a.m.; sets, 5:28 p.m. Moon rises, 8:11 p.m.; sets, 1.15 p.m.

WINNIPEG, MONDAY, OCTOBER 20, 1913.

ERY ABOUT DEATH

Pinned on Tree Tells e Body Was to be Found.

N VICTIM'S WRITING.

Adams Had Been Missing Two from His Vancouver Home.

ver, B.C., Oct. 18.—The mys- unding the death of William s, aged 19 years, whose body d yesterday in Stanley park, ing. Adams disappeared from two weeks ago and when his s found Saturday, with a half- tle of poison beside him, it ved he had committed suicide; of his face had been chewed some animal. The body was William Moorehouse. He saw nned on a tree: "Notice, please he dead body. Man lies under bout 200 feet back from here. lips of paper scattered in the his not was not in Adams' hand The slips of paper, too, ap- o have been of comparatively ate, though it is possible they ced there by Adams.

er theory advanced seems to orities to be more reasonable. that someone discovered the d, not wishing to be bothered ending an inquest, took the method employed of letting the now of the tragedy.

SIDENT HUERTA EMAINS AT HELM

Not Resign Nor Run Away, Declares—Has Plenty of ey For His Projects.

City, Oct. 19.—Provisional Huerta has not resigned, nor d from the capital. When

BIG ELEVATOR ANNEX SWAYS UNDER STRAIN

Foundation Gives Way Under C. P. R. Grain Warehouse At North Transcona

BUILDING IS IN RUINS

Contained Almost Million Bushels of Wheat---Machinery Is Wrecked

The annex of the huge C.P.R. internal terminal elevator at Transcona sank steadily for 12 hours on Saturday until it had subsided into the ground for 20 feet, at the same time tilting over to an angle of between 30 and 35 degrees. At midnight, when it was feared that the whole structure must collapse, owing to the tremendous strain placed on the concrete of which it was constructed, by the angle at which the building had canted to, the heavy cupola on top collapsed and, by relieving the strain on the foundation, brought the slipping to a stop. The huge storage bins in the annex were filled almost to their capacity of a million bushels of wheat, and, when the cupola fell, it destroyed the whole of the complicated apparatus for removing the grain from the bins to the work-house, so that the C.P.R. is now con-fronted with the problem of getting almost a million bushels of wheat out of a building tilted to an angle at which it appears as though it must collapse every moment, in time to have is shipped to the great lakes before the

AIR LINE TO WINNIPEG

Aviator Intends Flying from Minneapolis in New Machine.

HAD SUCCESSFUL FLIGHT.

Young Man Wishes to Prove to His Parents that His Biplane Is All Right.

(Free Press Special.)

Minneapolis, Minn., Oct. 19. — An aeroplane trip from Minneapolis to Winnipeg as a climax to months of industry is now the aim of A. T. Heine, aviator, who Friday afternoon made a successful flight at Fort Snelling with his biplane, built along Curtis lines. After a few more trips he plans to be off for Winnipeg this week. He says he is going to show his father and mother that he succeeded in the face of their disapproval and opposition. Heine Jr. says he knows no better way to demonstrate it than to land in an aeroplane just as far from his father's home as possible.

"The plane's behavior in Friday's flight convinced me I have as good a machine as I have ever ridden in" said Heine "and I have added features that add to its safety. I can easily make 75 miles an hour and if I fly at that rate only two hours a day, I can make Winnipeg easily."

Heine took a course in aviation from Glen Curtis. Immediately after he received his license he came to Minneapolis and set out to build his aeroplane. He plans to start a school of aviation here later.

TWENTY SOLDIERS KILLED IN WRECK

Troops Train Plunges Through Trestle Near State Line, Miss. —One Hundred Injured.

CANADIAN TROUBLE

Head Office Branches By Au

MAY ANNUL

Alleged that Emi gled Across Fr Disguised as W

London, Oct. 19. Daily News corre the operations of out Austria have Telegraphing yest Vienna office and fices of the Canad in the Austrian p and sealed by the cash account hoo been seized. The pany is suspended that the concessi It is said that the handed action is ening of he army of the Galician p tificial encourage with which compa

The Nieue Frei over 120,000 men service have been Canada during the

terday by the poli several other shipp and many arrests been discovered th had relays of autom costume to disguis ing the frontiers

...ey For His Projects.

...City, Oct. 19.—Provisional ...Huerta has not resigned, nor ...d from the capital. When ...National palace at 5 o'clock ...afternoon, he said he had no ...' doing either.

..."resign," said Gen. Huerta, ...to seek a resting-place 6 ...soil. When I flee the capi... ...be to shoulder a rifle and ...place in the ranks to fight ...

President Huerta's answer ...as to whether there was any ...n for the reports, which have ...culated in the capital and ...ir way to the United States. ...uerta looked the picture of ...l energy. Attired in a brand ...sack suit, he greeted The ... Press correspondent with ...ordiality, motioned him to a ...for a cigarette, and listened ...lanation of the motive for ...

...reported that I have fled," ...president. "You can see for ...at I am here at my post. ...I have resigned or intend ...an absolute falsehood. I ...ntention of resigning. ...the elections, which will be ...promised, indicate another ...esidency, I shall step aside. ...time you will find me here, ...with my promises to the ...ich are to re-establish peace, ...law if possible, but to re... ...peace."

...Has Plenty of Money.

...Huerta interspersed his ...anecdotes illustrative of his ...

...thing," said the president, ...money for my requirements, ...k that I haven't. Where did ..." "My secret," he responded, ...his chest with a satisfied ...that I have it."

...question of pacification, he ...that before the end of the ...government would have re... ...eason and made headway to... ...pacification of the state of ... "I have 8,000 men at Hipo... ...the president, "moving on ...and 2,000 more are proceeding ...n Zacatecas."

...to a suggestion that further ...re afloat to the effect that ...train proceeding to Torreon ...blown up, he exclaimed: ...nothing of the kind has ...

...otection For Foreigners.

...been said that all Americans ...ordered to leave the capital." ...ponse, Gen. Huerta made a ...of disgust. "What nonsense," ...d. "As I have repeatedly said ...came to the presidency, for... ...from Hottentots to the most ...ned, have received and will ...to receive every guarantee. ...likewise who obey the law ...thing to fear from me. 'Trans... ...must watch out. They shall ...shed through every means the ...ards."

...opinion of Gen. Huerta, these ...nal rumors have originated ...enemies of the administration ...capital and elsewhere for the ...hey would have with the re... ...ists.

...Huerta is working from 15 to ...s daily, sleeping at odd times, ...tigue overtakes him. He adopts

fronted with the problem of getting almost a million bushels of wheat out of a building tilted to an angle at which it appears as though it must collapse every moment, in time to have is shipped to the great lakes before the freeze-up. For a time it was feared that the structure must collapse altogether scattering the grain in all directions, but the wheat is now believed to be safe, unless in the event of a heavy rainstorm. The workhouse was not damaged.

For some time past, the soil at North Transcona has been known to be unreliable, serious caveins having occurred at the subway which the C.P.R. is constructing in the immediate vicinity of the elevator, but it was believed that the elevator itself was safe. Huge consignments of grain have been received at the elevator during the past two weeks, and the storage tanks, which have a capacity of a million bushels, were practically full. Though an official examination by engineering experts into the cause of the slipping has not yet taken place, it is believed that the great weight of grain in the tanks proved too great a strain on the foundation.

A few minutes before midday on Saturday, it was noticed that the annex was beginning to sink into the ground. During the first hour the elevator subsided one foot, the sinking being on the plum. To this circumstance is attributed the fact that the elevator did not topple over altogether, for, had it began to tilt immediately, is is believed that nothing could have saved the structure. As events turned out, it is believed that the foundations sinking evenly into the ground gave the annex sufficient additional steadiness to withstand the strain of the tilt.

After the first hour, the annex began to tilt over toward the west, sinking at the rate of two inches every five minutes. The sinking continued at this rate until 5.30 in the evening. During the next 15 minutes the sinking was only two inches, and, although the darkness prevented any further accurate estimates being made, it is believed that the rate of sinking continued steadily at the rate of eight inches an hour until midnight.

Situation Acute at Midnight.

By midnight the situation had become critical, it being feared that if the annex were tilted over at an angle any more acute, the entire structure would collapse. The belts, conveyors and other apparatus for transferring the wheat from the storage bins to the workhouse, were located on the top of the annex, and were covered by a heavy concrete cupola.

Was Great Crash.

When the annex had slipped to an angle of approximately 30 degrees, the entire cupola detached itself from the rest of the building, and crashed to the ground, with a roar like that of an explosion, which was heard for miles in every direction. With the diminution of pressure caused by the removal of

(Continued on Page Two.)

Troops Train Plunges Through Trestle Near State Line, Miss. —One Hundred Injured.

Meridian, Miss., Oct. 19. Twenty soldiers were killed and about 100 injured when a special troop train on the Mobile and Ohio plunged through a trestle near State Line, Miss., this afternoon. The casualty list is given in a report of the disaster by Division Superintendent Pighon, of the Mobile and Ohio, sent to the headquarters of the road in Mobile. All those killed were privates and members of the 39th coast artillery.

Reports from the scene of the wreck are meagre because of interrupted wire communication. Most of the dead and injured, it is reported, were taken to Mobile on a special train and another relief train was headed towards this city.

The 39th and 170th companies of artillery stationed at Fort Morgan, were taken to Mobile this morning, and at noon boarded the special Mobile and Ohio train bound for Meridian, where the soldiers were to participate in a fair. On the train were 179 officers and men.

Relief trains carrying physicians and nurses were sent from Mobile, Meridian and Whistler, Ala.

Ends Life on Fourth Attempt.

Vancouver, B.C., Oct. 19.—After having made three attempts on his life, Adolphe Marcand, a French-Canadian, committed suicide Saturday night in New Westminster with a rope improvised from a pillow-case and the lining of his coat. A few months ago he tried to end his existence by drowning in the bath tub in a hospital.

...terday by the po... several other shipp... and many arrests ... been discovered th... had relays of autom... costume to disguis... ing the frontiers a... and the police the... of these agents ... Pacific.

Anothe...

Vienna, Oct. 1... closed the offices ... Pacific Railway co... and throughout Au... ing all the books ...

This action was ... to the arrest on Th... Altmann, the ... connection with a ... assisted Austrians ... tary service by ... without passports.

Alexander Blan... Imperator Touris... also an agency of ... fic Railway was ar... this afternoon. The... domiciliary visits ... of eight other shi...

Sir T. Shaugh...

Milwaukee, Wi... tion of the Austr... closing all office... Pacific railway ... discussed by Sir T... in an interview he...

Sir Thomas is ... grain from the ... of the railway, ... he gives as reaso... that the steamship ... had unwittingly ... from Austria who... with government ... ing compulsory mili...

Alexander Blan... Imperator Touris...

(Continued o...

WHERE TO GO TODAY

8 a.m.—Dedicatory service at new St. Cuthbert's parish h... Elmwood. Supper and entertainment in evenin...
10 a.m.—100th Grenadiers' church parade, Westminster c...
2 p.m.—Eaton cup shoot, St. Charles rifle range.
6 p.m.—Dedicatory service and dinner at All Saints' Unite...
6 p.m.—Dedicatory service and dinner at All Souls' Unite...
8 p.m.—Liberal meeting, Chambers of Commerce.
8 p.m.—Theosophists, Manitoba hall.
8 p.m.—Thomas Richardson, Labor M.P., Trades hall.
8 p.m.—Concert, Central Congregational church.
8.15 p.m.—Dr. S. L. Krebs, lecture to Business Science ...

Theatres.

Walker, 2.30 and 8.15 p.m.—What Happened to Mary.
Winnipeg, 2.30 and 8.15 p.m.—A Butterfly on the Wheel.
Orpheum, 2.15 and 8.15 p.m.—Vaudeville.
Empress, 3, 8 and 9.30 p.m.—Vaudeville.
Strand, matinee and night—Vaudeville.
Victoria, 2, 3.30, 8 and 9.30 p.m.—Vaudeville.
Majestic, Starland and Monarch—Moving picture featur...

Sporting Events.

10 a.m.—Boys' Club races, ½, 2 and 5 miles.
10.45 a.m.—Y.M.C.A. 5 mile race; start west of Deer Lodge Vaughan Street
11 a.m.—Hunt Club meet, Agricultural College, St. James
1.30 p.m.—Winnipeg Driving Club races, Exhibition track
2.30 p.m.—Winnipeg Quoiting Club Tourney, John M. Kin...
3 p.m.—Senior Rugby—St. John's vs. Rowing Club, River...
3.30—"Bell" Cup final, Manitoba College, Scottish vs. ... grounds, Irish Cup (two games), Celtic vs. Uls...
8.30 p.m.—Boxing match, Amphitheatre rink—Freddy ... Saylor.
Arena Rink—Roller skating, afternoon and ...

Sinking of C.P.R. Elevator Annex at North Tran...

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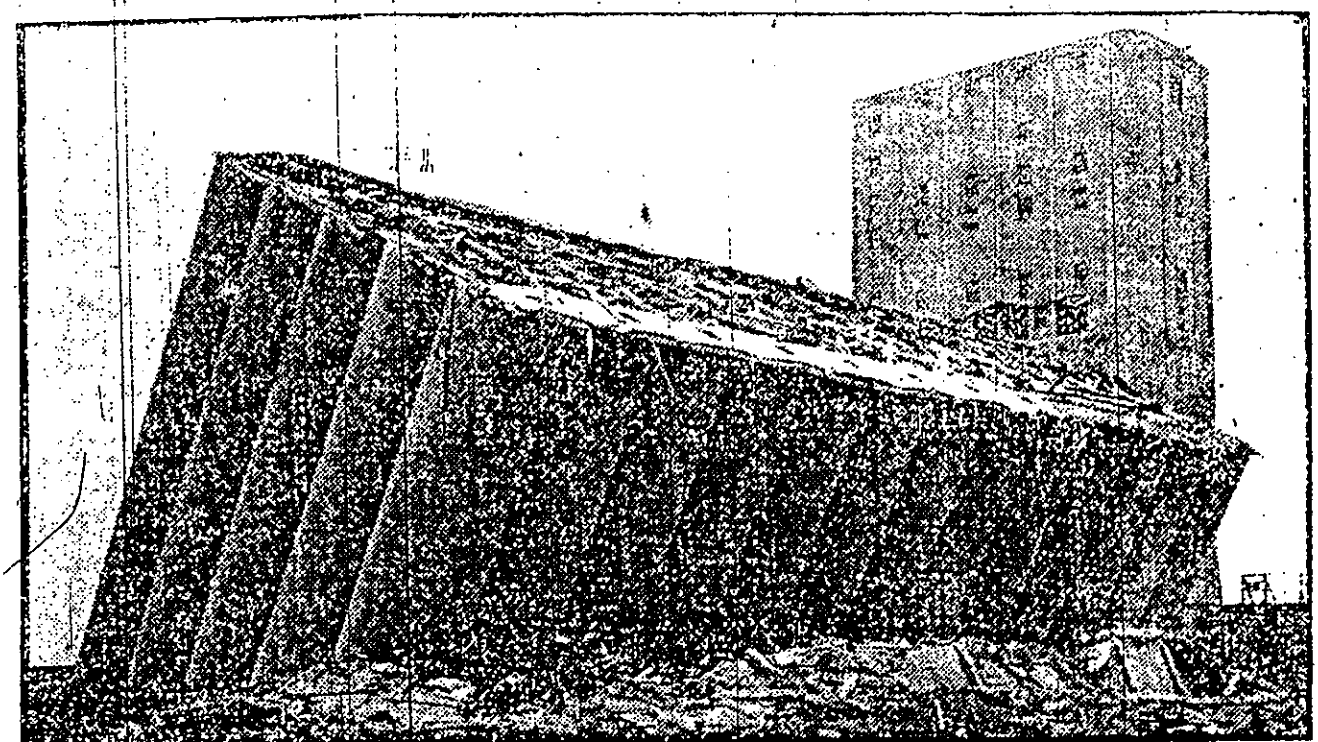
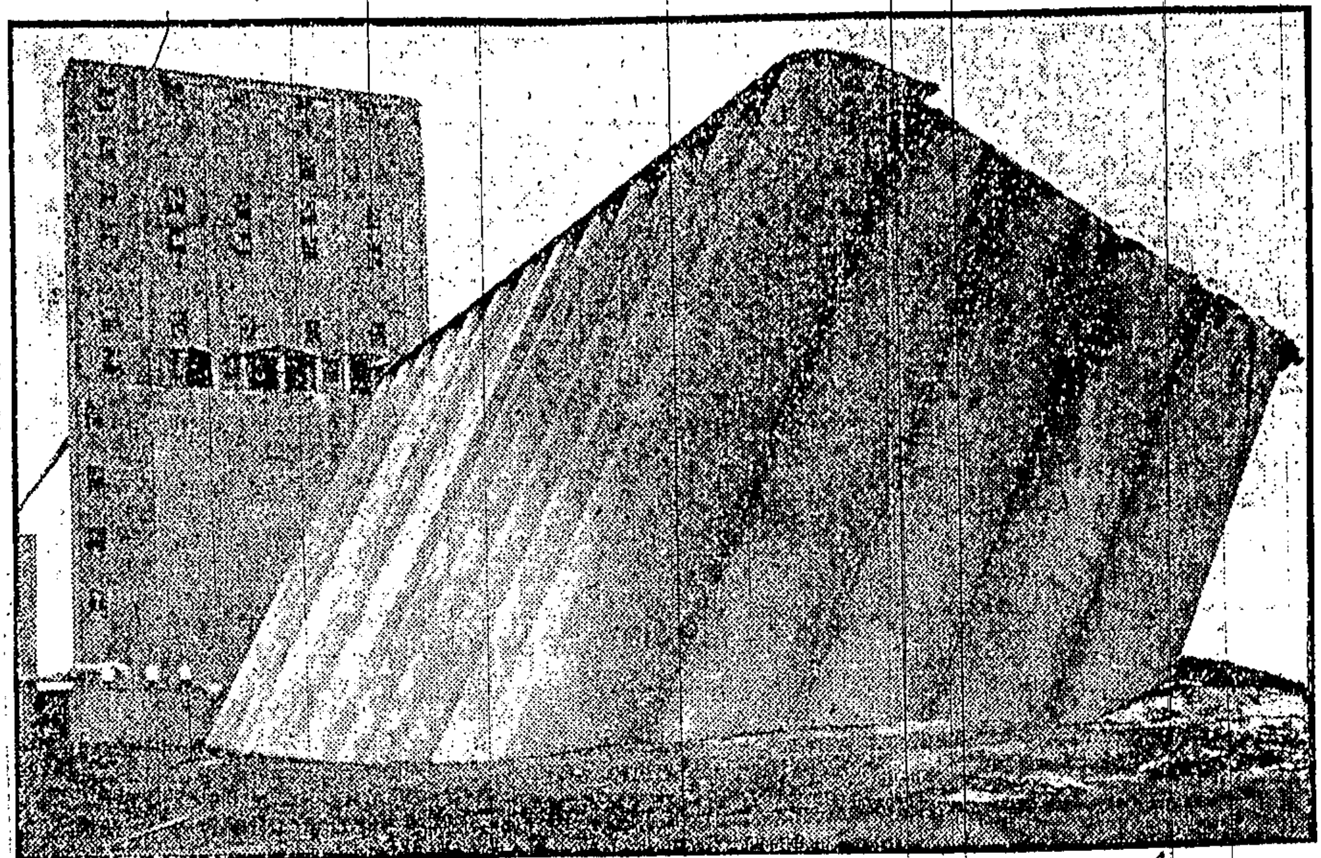
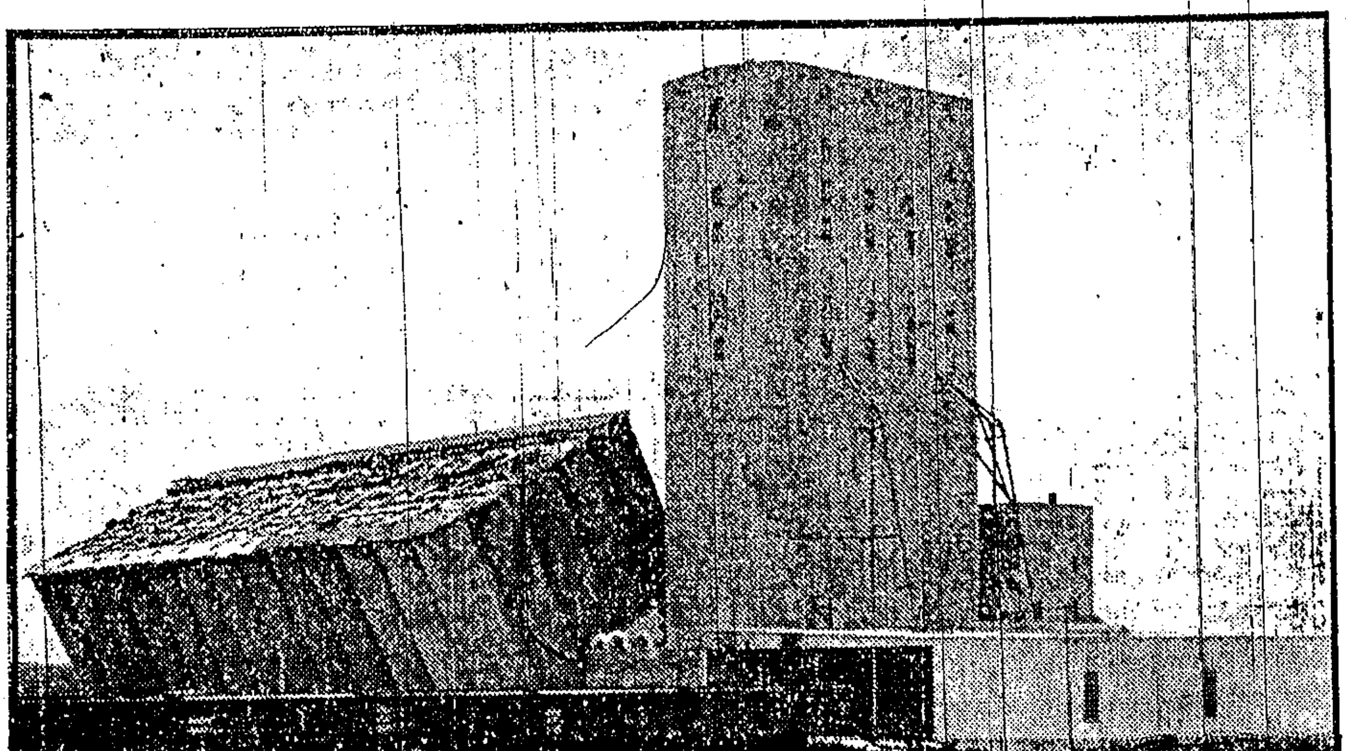
Sinking of C.P.R. Elevator Annex at North Transcona



This picture conveys a good idea of the disaster to the big grain storehouse on Saturday. The cupola crashed to the ground with a noise like a tremendous explosion, and the valuable machinery it contained was completely wrecked. The ruins of the cupola are seen lying beside the tilted concrete mass.

Two Views of the Wrecked Elevator at North Transcona

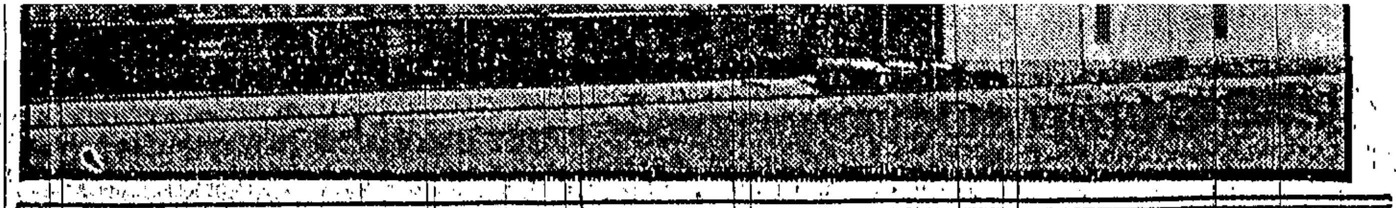




BIG ELEVATOR ANNEX

contained 65 large storage tanks and 48 smaller ones. The capacity of the annex was estimated at a million

Manitoba and throughout the west generally in improved methods of farming.



BIG ELEVATOR ANNEX SWAYS UNDER STRAIN

(Continued from Page One.)

the cupola, the foundations were able to resist the strain, and the subsiding ceased.

Remarkable Scene.

The scene presented on Sunday was a remarkable one. The huge annex, which is constructed of concrete, had swung over to an angle of from 30 to 35 degrees. At that angle, it seemed impossible that the structure could remain standing, yet it appeared as steady as though it were on its normal plane. As the annex canted over, it forced huge mounds of earth to the side on which it was reclining and the great volume of earth served as an additional support. While the foundations on the one side had subsided into the earth for a distance of 20 feet, the foundations to the rear were elevated by the canting of the building, and were brought almost clear of the ground.

Immediately it was noticed that the elevator was slipping, notification was given to the C.P.R. officials in Winnipeg, and a party immediately left for Transcona by special train. It was seen, however, that nothing could be done to stay the subsiding, and the elevator was left to work out its own course, the officials returning to the city.

Excitement at Grain Exchange.

The slipping of the elevator caused the greatest excitement on the Grain Exchange, the majority of the members having consignments in the storage bins. Throughout the afternoon, grain dealers kept constant watch on the elevator through telescopes and field glasses and it was stated that, even at that distance, an appreciable difference could be noticed in the position of the elevator every half hour.

One of the most remarkable incidents in the whole remarkable accident, lay in the fact that no men were in the annex when the slipping commenced. Had the accident occurred at any other hour of the day, the structure would have been crowded, but, just before the slipping started, the men employed had left work for the lunch hour.

Problem to Ship Grain.

The principal problem now confronting the C.P.R. is to get out the grain and ship it east, with the least possible delay. W. Brook, who has charge of the shipping of grain from the elevator, made a preliminary examination yesterday, as a result of which he stated that so far as he could learn no damage had been done to the grain. It was possible, he said, that some of the wheat in the foot of the bins might be injured by water, but he did not consider that any serious damage had been done. The principal danger to be faced lies in the fact that the falling of the cupola has broken the concrete covering which served as a protection for the storage bins. In the event of a heavy rainfall, the water would soak through the holes contained 65 large storage tanks and 48 smaller ones. The capacity of the annex was estimated at a million bushels.

Had Taken Precautions.

Every possible precaution against such an accident was taken before the elevator was constructed, according to J. G. Sullivan, chief district engineer for the C.P.R. Since the erection of the elevator Mr. Sullivan stated, additional proof of the unreliability of the soil at North Transcona had been afforded by the slip [illegible] of the subway, but, at the time it was believed that the earth was solid. Within a hundred feet of the elevator, he stated, a 500 foot well had been dug and had disclosed no soft soil. In addition, the elevator had been tested with twice the weight under which it collapsed. The reason for the slip was given by Mr. Sullivan as lying in the existence of soft soil beneath the elevator, which had hitherto been unsuspected.

The cost of the collapsed building was between $140,000 and $150,000, and Mr. Sullivan stated that the insurance did not cover such a contingency. He added that it was doubtful if any effort would be made to restore the annex, the cost involved being practically as great as the original price. Barnett and McQueen were the contractors.

HEARS FORMER PASTOR

Augustine Congregation Addressed by Rev. R. G. McBeth.

Rev. R. G. McBeth made a welcome appearance to his former church, when he preached yesterday morning and evening at Augustine. The preacher took for his text Psalm 103: 2-5, "Bless the Lord, O my soul, and forget not all His benefits. Who forgiveth all thine iniquities; who healeth all thy disease; who redeemeth thy life from destruction; who crowneth thee with loving-kindness and tender mercies."

Rev. McBeth stated that the psalmist probably wrote this psalm in a fit of depression, to which he, like all other men, were apt to be subject at times. He gave illustrations of great men in history who had at various times passed under fits of despondency. David wrote this psalm in such a fit. But his knowledge of human nature taught him that the best way to escape from fits of depression was to recall reasons for thanksgiving to God. Therefore he began to count his blessings, but found them so numerous that he had to sum them up throughout the whole psalm. And in order that he might bear them in mind he wrote this psalm. Taking up the various scenes of the psalm the speaker dwelt first on the forgiving of iniquities, at the courtroom and palace of the king. Everyone, he said, was guilty of breaking the law of God, and everyone, urged by their conscience, would face their guilt and throw themselves on the mercy of the court. God as judge would forgive. Then he dwelt on the great dispensary of God. The gospel of Christ was the gospel of good health. Sin was a deadly thing. The speaker closed his sermon by dwelling on the other scenes in the psalm. Tonight he will deliver an address on Florence Nightingale at Augustine.

Manitoba and throughout the west generally in improved methods of farming.

Presentation to President.

The usual evening entertainment in the lecture hall of the Industrial Bureau was marked by a pleasant function On behalf of the directors of the show, Commissioner Roland presented J. Bruce Walker, the chairman, with a handsome silver-mounted umbrella and cane. In making the presentation, Mr. Roland stated that much of the success of the show was directly attributable to the energy and enthusiasm of Mr. Walker. While the other directors had regarded the show more or less as an experiment, he said, Mr. Walker had always been absolutely confident of its success, and events had proved that his confidence was not misplaced. In returning thanks, Mr. Walker stated that, if the show had done anything to increase the good name of Winnipeg or to further the cause of agriculture in the province, he was amply compensated for his work.

Throughout the closing day of the show, the attendances were again very large, nearly 5,000 persons passing the turnstiles. It is estimated that the exhibition has been seen by over 25,000 people since the opening.

All the competitive exhibits were offered for sale, and met a ready demand, every article being sold early on Saturday.

PINCH OF PROSPERITY

Faith Weakens When Need for Effort Ceases, Says Rev. A. A. Shaw.

On the subject of divine power, Rev. A. A. Shaw, of Cleveland, and formerly of the First Baptist church, Winnipeg, preached the Thanksgiving sermon in that church yesterday morning. In opening his address Rev. Mr. Shaw called attention to a little booklet by Alfred Russell which proved that more progress had been made in the nineteenth century than since the world began. This great advancement had been made because control of and were brought almost clear of the ground.

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Explaining the probable method to be adopted in salvaging the grain, Mr. Brook stated that the tanks would be tapped and a temporary convey fitted to workhouse. By this means, he stated, the grain could be shipped almost as quickly as by the regular apparatus, which was destroyed by the falling cupola.

Complete investigations have not yet been made into the extent of the damage, but Superintendent McLean stated that it was improbable that the annex would ever be used again. It was a remarkable fact, he stated, that, despite the great angle at which the annex had tilted, there was not a single crack in the concrete except in one small place, and that was caused not by the strain, but by the falling cupola. The workhouse, the superintendent stated, was uninjured, and could be used, with the addition of a new annex.

The elevator at Transcona was only erected last year, and was used for the first time last fall. It was of the most up-to-date type, with large workhouse and annex behind. The annex itself was constructed wholly of concrete and

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APPLE SHOW CLOSED

Over 25,000 Persons Visited Exhibition Financially Successful.

The first Canadian Land and Apple show was brought to a close on Saturday evening, after an entirely successful ten days session. The directors are entirely satisfied with the results achieved. In addition to having been a financial success, and to having proved a very valuable advertisement to Winnipeg, the show has been of great value in interesting farmers in

All the competitive exhibits were offered for sale, and met a ready demand, every article being sold early on Saturday.

PINCH OF PROSPERITY

Faith Weakens When Need for Effort Ceases, Says Rev. A. A. Shaw.

On the subject of divine power, Rev. A. A. Shaw, of Cleveland, and formerly of the First Baptist church, Winnipeg, preached the Thanksgiving sermon in that church yesterday morning. In opening his address Rev. Mr. Shaw called attention to a little booklet by Alfred Russell which proved that more progress had been made in the nineteenth century than since the world began. This great advancement had been made because control of

Mr. Chase.

columns and beams were chamfered liberally. These points are not shown accurately in the drawings.

The vertical stirrups, to which exception is taken, were spaced so that the shear on the concrete would not exceed 60 lb. per sq. in. Failure by diagonal cracking is resisted effectively by vertical stirrups, even if their exact disposition may not be subject to rigid analysis.

In commenting on the sidewalk bracket connection to the bascule girders, Mr. Godfrey has drawn his conclusions from insufficient information. Details of this connection are not given in the paper. In addition to the two rivets in tension, which are shown in the girder view, Plate XXXII—which Mr. Godfrey guessed were all that held the bracket up—there are from five to six rivets in shear, connecting a top horizontal plate to the top flange of the grider. The maximum stress in the top chord of the cantilever bracket is only 15 000 lb.

Mr. Ritchie has added several facts of interest which supplement the writer's account of the operations of his company. The construction of the bridge in two longitudinal halves certainly added greatly to the cost and length of time required to construct the complete bridge. The Cherry Street Bridge was, as he intimates, a storm center in Toledo's political life from its inception until its completion, and hard-won legal victories could not be jeopardized by changes in the plans of construction which otherwise might have been made as their desirability developed.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

TRANSACTIONS

Paper No. 1363

THE FAILURE AND RIGHTING OF A MILLION-BUSHEL GRAIN ELEVATOR*

By Alexander Allaire, M. Am. Soc. C. E.

With Discussion by Messrs. David Gutman, W. R. Phillips, and E. P. Goodrich.

Synopsis.

The object of this paper is to present to the attention of the members of the Society the history of a rather unusual engineering feat. The size of the building to be straightened, the angle to which it had tipped, and the weight to be handled, all combined to make the work unique.

The restoration of the elevator to a working condition may be divided into four parts:

1.—Making safe the foundations under the workhouse—a structure resembling a tall office building, resting on a very small base;

2.—Straightening the binhouse, a structure having an area of 15 000 sq. ft.;

3.—Providing this binhouse with a new and adequate foundation; and

4.—The renewal and repair of those portions of the original buildings which had been broken or deranged at the time of the failure.

This paper treats of the first three.

* Presented at the meeting of February 2d, 1916.

The Canadian Pacific Railway, which serves as the principal outlet from Western Canada, in 1911, found it imperative to provide relief for its Winnipeg Yards, which were yearly becoming less and less able to meet the demands on them. During October, November, and December, Canada's immense wheat-producing provinces of Manitoba, Saskatchewan, and Alberta ship their grain to the United States and Europe *via* the Canadian Pacific Railway to Fort William, where it is loaded on the Lake boats. To relieve the congestion, a cut-off was made, running approximately 3 miles north of Winnipeg, and one of the largest railroad gravity yards in the world was built at North Transcona, 7 miles northeast of the city. As an additional aid to the speedy shipment of the grain, a million-bushel elevator was constructed at the North Transcona Yard. This is to serve for storage during the "peak" periods, and relieve the car shortages by sending empties back West several days earlier than would be possible, should these cars have to go as far as Fort William.

The country in the vicinity of Winnipeg is flat, and the character of the soil is quite uniform throughout the district. First there is a stratum of about 2 ft. of heavy black loam, below which there is reddish gray clay, 5 or 6 ft. in thickness, and generally water-bearing, and this gradually changes to a blue clay extending to a depth of approximately 40 ft. below the surface, where it changes suddenly to white clay interspersed with limestone boulders. Underlying the white clay is found a stratum, averaging about 30 in. thick, of shattered limestone which in turn overlies the limestone rock. At Transcona this rock varied from 53 to 55 ft. below the prairie level.

Usually, the blue clay found at the depth from 7 to 8 ft. below the surface is very firm, and is capable of carrying a load of from 3 to 4 tons per sq. ft. As a result, it has been the practice in the Canadian Central West to use floating foundations. In building the elevator, therefore, this general custom was followed.

The ultimate loading of the clay under the mat was calculated at 3.3 tons per sq. ft. As a matter of precaution, soil-loading tests were made at different points on the site. From the result, it was thought that the customary slight initial settlement might be experienced, but that nothing more serious should occur.

The general layout consists of four units; a workhouse, 70 by 96 ft. and 180 ft. high; a binhouse, 77 by 195 ft. and 102 ft. high; a dryer-house, 18 by 30 ft. and 60 ft. high; and a boiler-room equipped with two 100-h.p. locomotive boilers. The workhouse rests on a reinforced concrete floor, 30 in. thick, and is a reinforced concrete structure, with brick curtain-walls as the top is approached. The basement of the building is 16 ft. high. In it are the belts for transferring the grain from the cars to the conveyor-boots, and from the binhouse to the workhouse. The ground, or prairie-level, floor is occupied by the cleaning and drying machinery. Above these machines there are fifteen bins, 13 ft. in diameter and 70 ft. high, above which again are several floors carrying the weighing machines, etc. The floor, machinery, and wall loads above the second floor rest on twenty-four interior columns, placed in four rows of six columns each. The interior columns carry a load of about 800 tons each; the exterior ones carry about 500 tons each. Fig. 1 shows the workhouse in plan and elevation.

The binhouse, immediately north of the workhouse, consists of sixty-five circular bins arranged in five rows of thirteen each. These bins or tanks are 14 ft. 4 in. in diameter. The diamond-shaped spaces formed between the circular bins are also used for storage, each having a capacity of about 5 000 bushels. The bins, 92 ft. high, are surmounted by a cupola which houses the top conveyor and trippers, and extends the full length of the structure. With this the tanks are filled with grain. The bin walls are of concrete, 6 in. thick, with the customary reinforcement. Below the 12-in. reinforced concrete slab, which constitutes the bin bottoms, there are four conveyor-belt tunnels, 7 ft. wide, running the full length of the structure. These tunnels are formed by walls 16 in. thick and 7 ft. high, which rest on a 2-ft. mattress of concrete. Transverse to the main tunnel walls there are 15-in. cross-walls approximately 15 ft. from center to center. These are under the bin contacts. The tunnel or cross-walls were not bonded to the floor or bin slabs, and this constituted a considerable hazard during the straightening.

Excavation for the structure had been made to a depth of 12 ft. below the prairie level.

The dryer-house had broad footing courses under the walls. Although it was deemed advisable, later, to underpin this building, this presented no difficulty.

The Canadian Pacific Railway commenced storing grain in the binhouse in September, 1913. Considerable care was taken to regulate

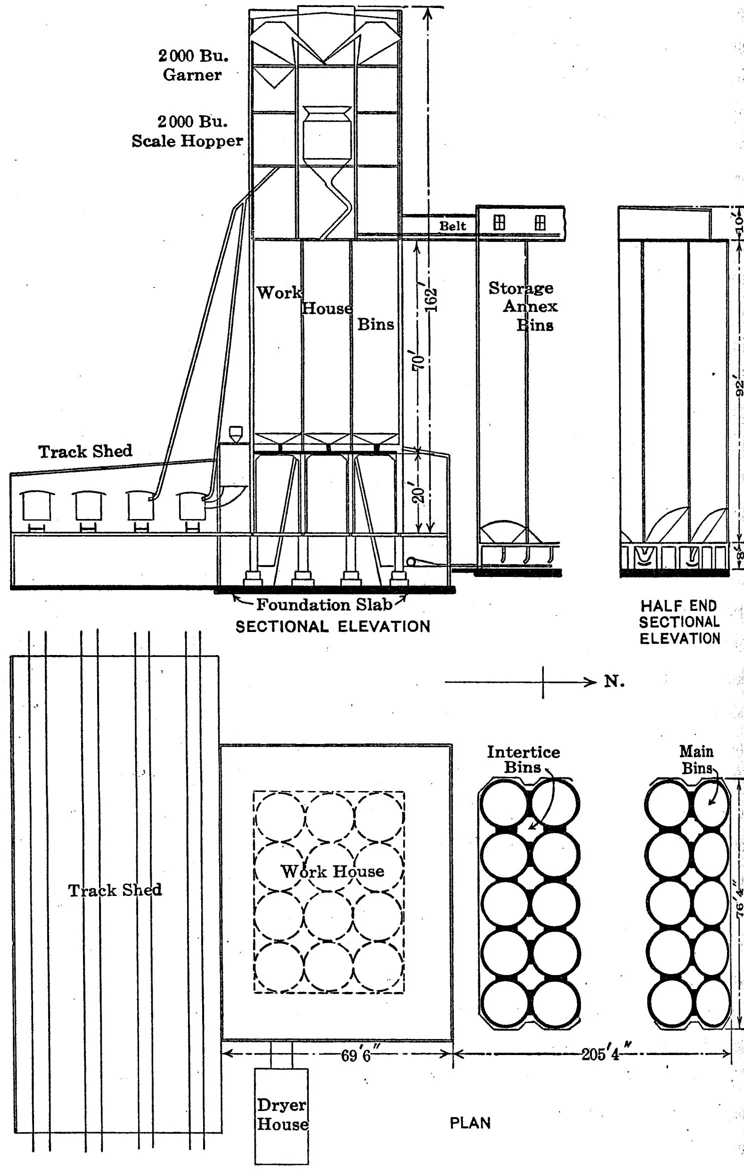
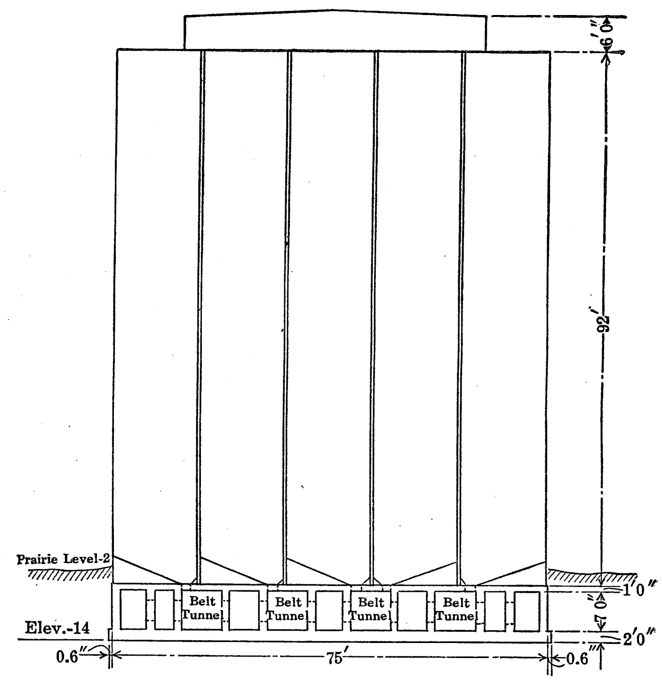


FIG. 1.

the filling of the different tanks so as to distribute the load uniformly. Settlement began on October 18th. The bins at this time contained about 875 000 bushels of wheat. A vertical sinking of 1 ft. occurred within an hour after the settlement was first noted. This was followed by an inclination toward the west, which increased until, at the end of 24 hours, the binhouse rested at an angle of 26° 53′ from the vertical.



ORIGINAL POSITION
SECTION THROUGH BINS—LOOKING SOUTH

FIG. 2.

An examination showed the east side to be 5 ft. above, and the west side 29 ft. below, the original position; the building was also approximately 4 ft. lower at the north than at the south end. An upheaval of 5 or 6 ft. of the ground surrounding the bins occurred during the settlement. At an angle of 26° 53′ the center of gravity of the loaded structure had moved over almost to the low edge. The upheaval and the compacting of the soil along the west side saved it from completely falling over. Above this, on the west side, 52 ft.

of tank overhung the ground. This overhanging load, due to the grain-filled tanks, was considerable.

A careful examination was made of the structure and, remarkable as it may seem, practically no shear cracks were found. It was decided, therefore, that the first problem was to save the wheat. This was done by tapping the most westerly row of tanks at approximately the ground level and bleeding out the grain upon a belt conveyor parallel with the line of the tanks. When the tanks of this outer row were emptied down to the ground line, holes were tapped in the next row, and the grain was spouted to the belt conveyor, and so on from tank row to tank row. The hazard of this operation will be realized when one considers that it was difficult to calculate what stresses were set up, due to the inclined position of the bins and the change in loading as the tank rows were emptied in succession. Added to this was the menace, at the top of the structure, of the remnants of the cupola, part of which had fallen to the ground during the settlement. To remove the grain below the ground level, a sheeted pit was excavated at the north end. A conveyor leg was placed in this pit, with the discharge end emptying upon the belt parallel with the west side of the bins. The tanks were emptied by spouting the grain to the tunnel belts, which, being reversed, carried it to the conveyor leg. It is worthy of note that, despite the handicap of the dangerous working and the fact that part of the conveying apparatus had to be obtained from Chicago, all the wheat was removed in less than 3 weeks from the time of the failure, and at a cost of less than 1 cent per bushel.

Immediately after the failure an examination was made with boring machines, and it was found that though the rock over the greater area was at a depth of from 52 to 55 ft. below the prairie level, an unusual condition existed along about one-half of the length of the east side of the bins—a ridge of boulders some 12 ft. higher than the rock being encountered. This was the reason for the tipping over, for, as the initial vertical settlement took place, resistance was offered along the east side, probably through compacting the soil above the ridge of boulders. This produced a tendency of the building to cant to the west. The heavy load caused the clay to flow and the structure to settle.

Fortunately, the workhouse was only slightly affected by the movement of the binhouse. Only a few cracks showed in the north shed wall, no settlement taking place in the building proper.

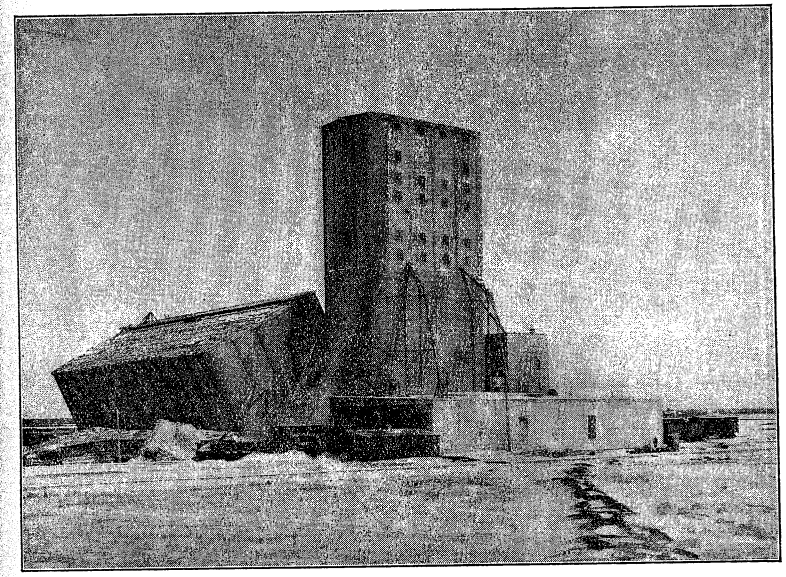


FIG. 3.—VIEW OF ELEVATOR AFTER SETTLEMENT.

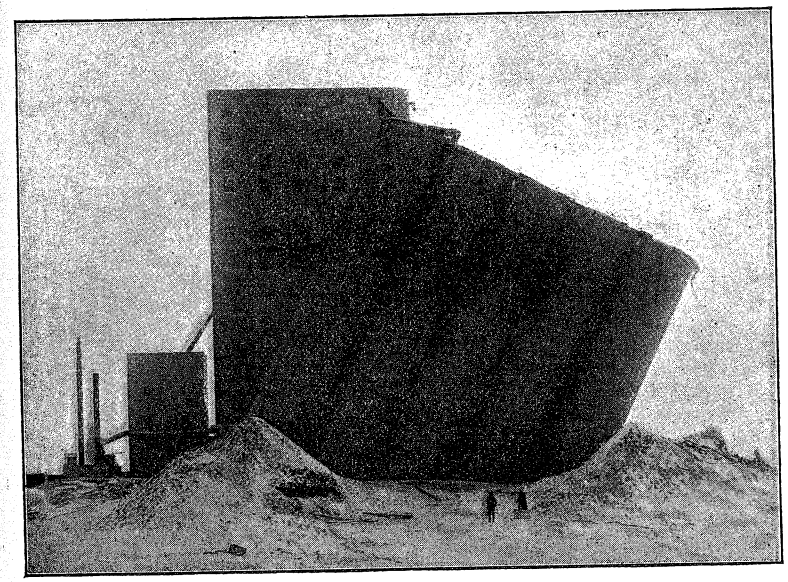
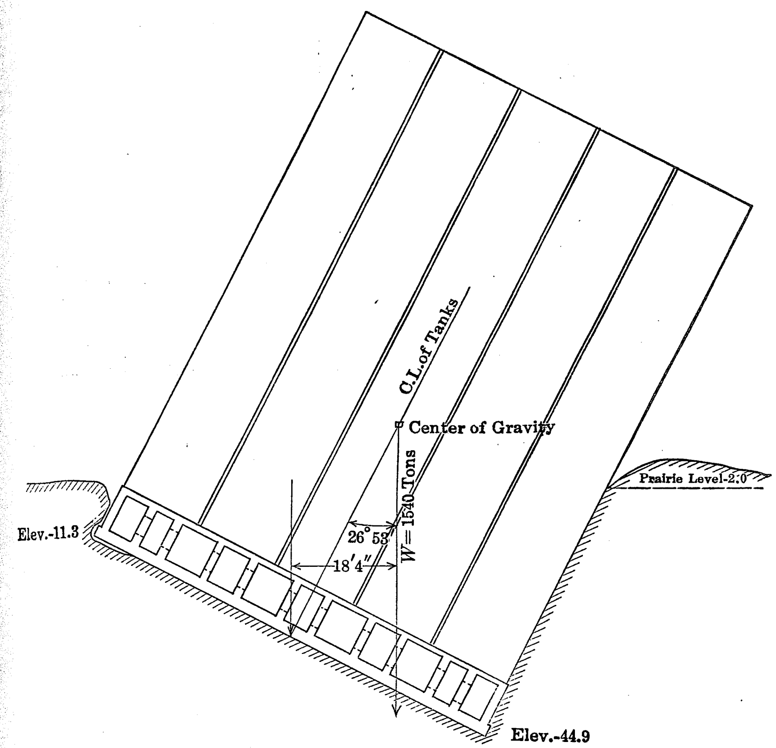


FIG. 4.—VIEW OF ELEVATOR AFTER SETTLEMENT.

In December, 1913, The Foundation Company, Limited, of Montreal and Vancouver, submitted a plan to the engineers of the railroad for underpinning the workhouse to rock, as it was feared it might fall. The plan was approved, and work was started immediately. In general, the plan followed was the sinking of a pier under each column of the building. Because of the heavy loads, the height of the structure, and its small base, it was necessary to shore the building before starting the underpinning operations at the twenty-four columns.



SECTION THROUGH BINS AFTER SETTLEMENT

FIG. 5.

The shoring consisted of pushers from twenty small piers placed outside the building columns, as shown by Fig. 6. On account of the uncertainty of the condition of the material through which the piers would have to be carried, it was decided to eliminate any risk by sinking the first six with cast-iron shoring cylinders. The cylinders

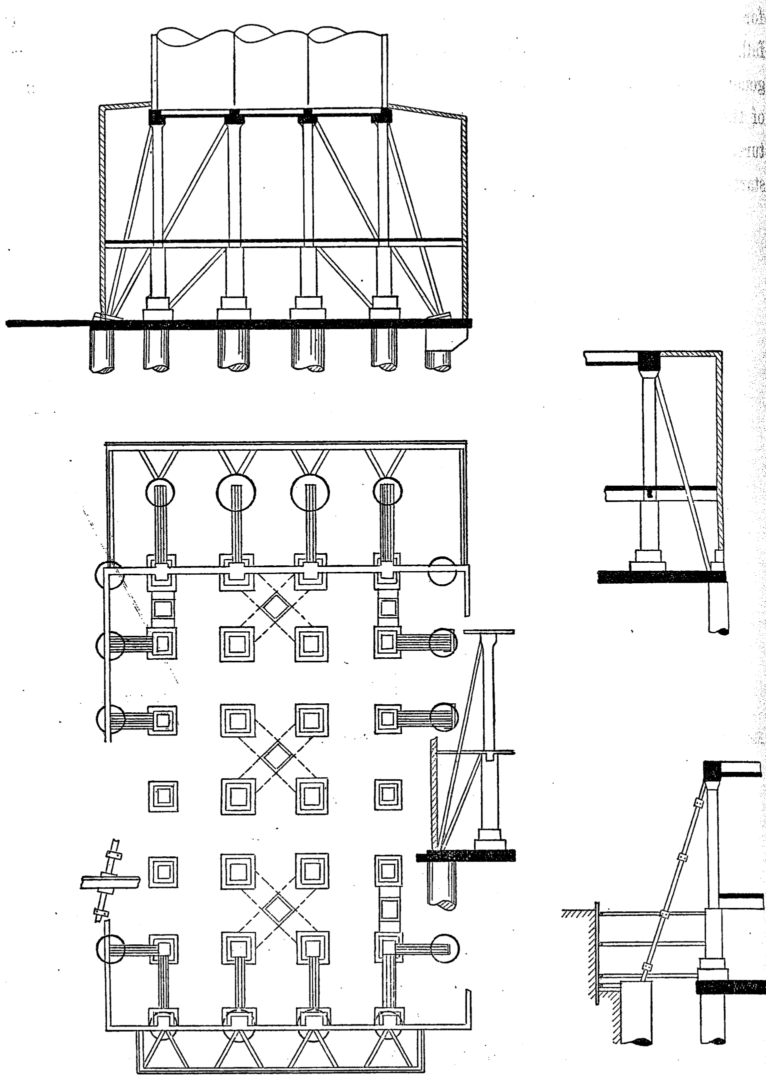


FIG. 6.

were 4 ft. in diameter, and built up in 3-ft. sections. Flanges were cast at each end of the sections for making connections. The one exception to this was at the bottom section, on which one flange was omitted, the unflanged rim being used as a cutting edge.

The shoring piers on the south and east sides were placed under the shed walls. Work on these was started by cutting a hole 4 ft. 6 in. square through the concrete floor, then drifting to the site of the pier. This excavation was sheeted, and the first section of the cylinders was placed in position. As the earth was excavated from within the shoring cylinders, the latter were jacked down by 100-ton jacks. When the top of each cast-iron section reached the ground level, another section was bolted on, and the operation was repeated. As a result of sinking six wells at different points throughout the area of the building it was decided that the Chicago well method, using wells 5 ft. in diameter, could be adopted.

The wells, with the exception of those on the south and east sides of the building, were placed clear of the walls. On the north side the work was carried on by cutting through the mat and sinking the wells immediately under it. At the west side of the building a sheeted pit was dug, approximately to the mat level, and the wells were started from this elevation.

Considerable water was encountered in sinking a majority of the shoring and underpinning wells. This was handled by No. 3 Pulsometer pumps, in some instances as many as three being required in a single well.

On each completed shoring cylinder there was placed a heavy timber shore. These were built in most cases of six 12 by 12-in. timbers, 40 ft. long, tied together with bolts and plates. Each pusher was heeled on oak blocking, placed on the top of the shoring pier at the basement floor level. Passing through an opening cut in the floor of the workhouse at the prairie level, it engaged an oak header, reinforced on two sides with 12-in. channels, the header being let into the column just below the bin floor by notching out the mushroom top, as shown by Fig. 7. Supplementing this was another pusher, extending from the shoring pier to the wall column at the first floor, and from this floor to the top of the interior column at the second-floor level.

Similar shores were placed on the shoring piers on all four sides of the building. These, combined with the columns, bin floor, and reinforced mat, formed a truss arrangement by which the load of the building was largely transferred to the shoring piers. Later, while the interior piers were being sunk, timbers (shown by the dotted lines) were put in to take the load from the interior column bases.

All the shoring having been placed, the work of sinking the 5-ft. wells under the wall columns and the 6 ft. 6-in. ones under the interior columns was commenced. The columns are 15 ft. from center to center. To gain access to the under side of the exterior columns, holes 3 ft. 6 in. square were cut through the concrete floor mat between pairs of columns. Drifts 4 ft. wide and 6 ft. deep were driven in the direction of each column. As a well site was reached, the space was enlarged to a circular area slightly greater than the outside diameter of the well lagging. These drifts or tunnels were sheeted to prevent loss of ground, and afforded the only means of communication with the wells. The excavated material was hoisted in light galvanized-metal buckets which traveled on curved tracks or skidways, necessary on account of the center of the top of the underpinning well and opening in the floor being off center by 7½ ft. At the basement floor level the spoil was dumped into wheel-barrows, which were wheeled to a motor hoist, raised to the prairie level, and wasted at a short distance.

Prior to opening up the wells at the workhouse, levels had been taken on the column footings. As the piers were sunk, check levels were taken at regular intervals. As a further check on any movement, a plumb-bob weighing 280 lb. was suspended from the roof of the building down through an elevator shaft. The total distance was about 160 ft. The plumb-bob was immersed in a tank of water which was prevented from freezing by an electric coil. The position of the bob was noted every day.

Wells under the interior columns were approached in the same manner, except that the holes through the floor mat were cut centrally with respect to four columns, and handled the wells under them by drifting to each site in turn as one after another of the wells was completed. The distance from the center of the floor opening to the center of each underpinning pier in this instance was 10 ft. 6 in. Special curved skidways were used for these wells also. In all cases single-drum hoists, electrically operated, and having a rope speed of 130 ft. per min., were used to elevate the excavated material.

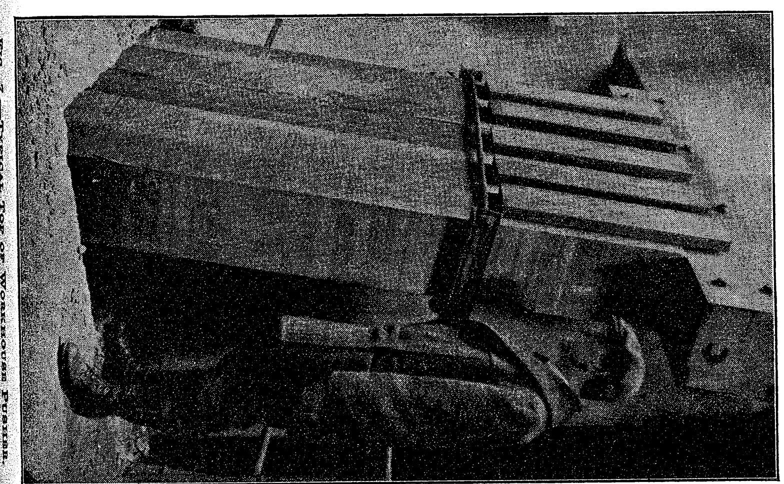


FIG. 7.—TYPICAL TOP OF WORKHOUSE PUSHER.

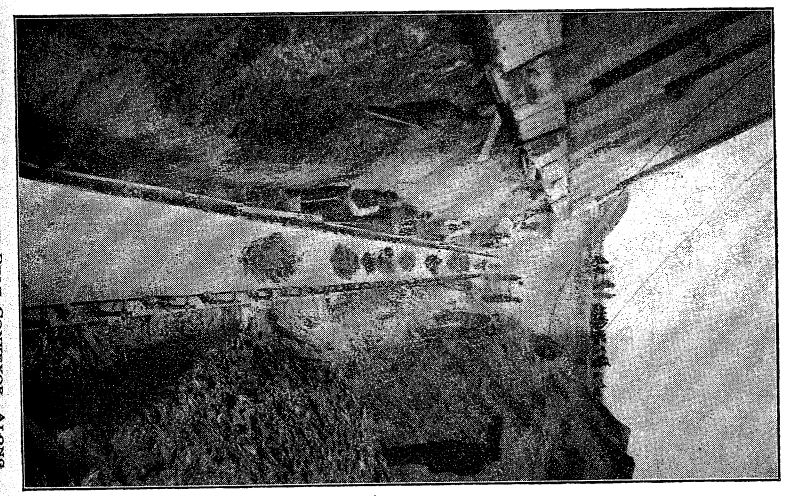


FIG. 8.—TRENCH, WITH BELT CONVEYOR, ALONG BACK SIDE OF WORKHOUSE.

Concrete, up to the top of the wells, was deposited through light 8-in. galvanized-iron pipe, cut to short lengths to accommodate them to the necessary curvature. Above this elevation, and up to within 8 in. of the under side of the columns, concrete was shoveled into place behind forms, the near side of which was built up as the concreting progressed. After this concrete had set sufficiently, the remaining 8 in. were finished both by grouting and by ramming in fairly dry concrete.

The workhouse operations were completed by about the beginning of June, 1914.

In the latter part of Feburary, 1914, permission was given to The Foundation Company, Limited, to proceed with the straightening of the binhouse. When the vertical position had been reached, it was planned to underpin it also by concrete piers to rock, these piers to be placed under the contact points of the tank walls in longitudinal rows. As a matter of economy, it was decided not to attempt to raise the building to its former elevation, but to straighten the structure by rotating it about the low edge; this was to be accomplished by excavating under the east or high side, and lowering it to the level of the low or west side. This meant that the mat in its final position would be approximately 38 ft. below the prairie level. As this was below the ground-water line, it was proposed to water-proof it.

The binhouse when empty of grain weighed 20 000 tons. Under the lower or west edge of the mat fourteen piers were sunk to rock and concreted. It was the intention to block the building off these piers to form a fulcrum, about which it would rotate as the excavating was done under the high side. The clay actually under the mat was removed in two ways: First, by working from under the high edge of the mat; and secondly, through holes in the mat. The former necessitated the excavation of a trench paralleling and flush with the east edge, as shown by Fig. 8. This was carried to a depth of about 8 ft. below the mat edge, was approximately 10 ft. wide at this point, and sloped back on the side away from the structure to the natural angle of repose of the soil. Drifts were driven from the trench toward the west under the mat. In the trench was placed a belt conveyor which discharged upon another belt, at the north end of the building, running at right angles to the first one. This second belt emptied into a hopper from which the earth was hauled by team and spread over the prairie. The holes in the floor were in the compartments formed by the cross-walls between those of the main tunnels. The spoil passed through the holes and was handled by two conveyor belts, one in the first and the other in the third tunnel from the east. These belts discharged upon the transverse belt, previously mentioned as running along the north end. The gradual settlement of

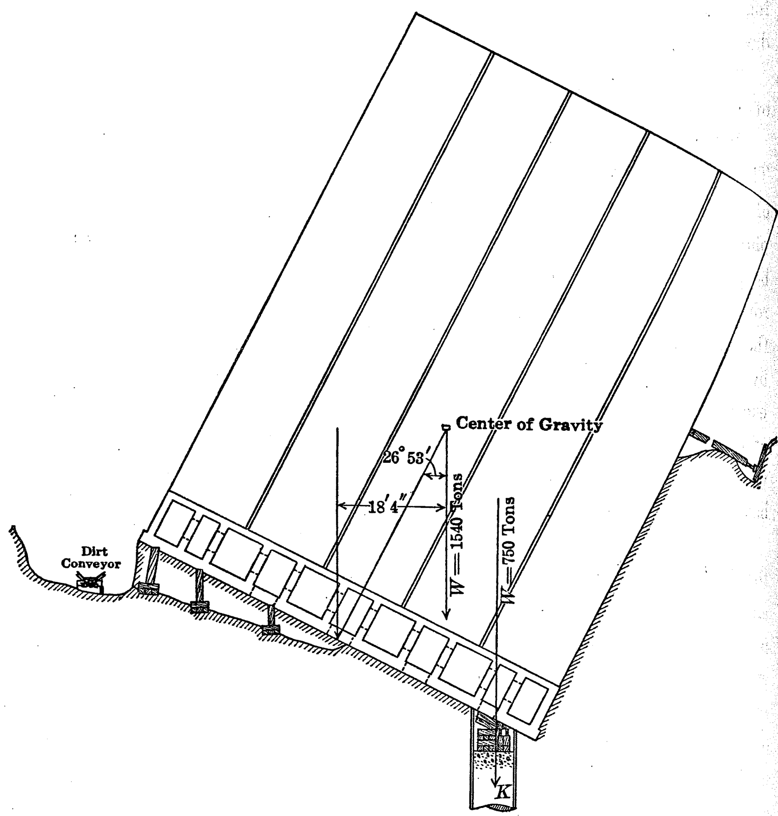


FIG. 9.

the high side was to be accomplished by weakening the core of earth left between the drifts. In driving the drifts, the earth wall between them was narrowed from the bottom up, the intention being that, as the building dropped, this core would offer increasing resistance to the movement, and that this movement could be controlled by removing the earth from the sides of the cores.

The fourteen piers under the low side, known as the *K* row, were placed on the longitudinal center line of the thirteen tanks, as shown by Fig. 9. They were placed under the contact walls and at the ends. Access to the sites of these wells was secured by cutting holes, 3 ft. 6 in. by 4 ft., through the mat in the compartments between the tunnels proper. The experience gained in sinking the workhouse wells showed that a large quantity of water would have to be handled in sinking the wells of the *K* row. Preparations for doing this were made by lagging down a large sump with 10 by 10-in. timbers at the north end of the building. This sump was carried to the depth of the low corner of the mat. It was placed inside the excavation which had been made to put in the grain leg and also the transverse belt conveyor used in connection with handling the spoil. In the sump was erected a duplicate set of electrically-driven centrifugal pumps and steam-driven piston pumps. Each set had a capacity of 1 200 gal. per min.

In sinking the wells of the *K* row, as well as the others described later, the work was performed under considerable difficulty, owing to the 27° inclination of the mat and the confined working space. Spoil from the different wells was raised by hand windlasses. The limited space and the small quantity of material to be handled would have made any mechanical arrangement more expensive.

The depth of sinking at the *K* wells was only 14 ft. Lagged wells of the Chicago type, 7 ft. in diameter, were used. The material was found to be very firmly compressed; in some instances the blue clay had been driven into the white stratum. The white clay, instead of being soft, as it had been found at the workhouse, was squeezed dry and hard, so that it was usually necessary to pick before shoveling into the buckets. No water was found until the shattered limestone stratum, immediately overlying the rock, was reached.

Not only was a large quantity of water found in the *K* wells, which from No. 1 to No. 6 came in at the rate of 1 150 gal. per min. in each well, but, in addition there was the danger that the level to which it would normally rise was approximately 14 ft. above the mat at the openings. This made imperative the duplicate pumping system at the sump. The insufficiency of room further aggravated the water troubles.

To do away with the multiplicity of piping of the small steam units and the almost unbearable heat from them, belt-driven, electrically-operated centrifugal pumps were erected immediately above the wells. These were placed on the main tunnel floors, and independent holes for the pump suctions were cut through the mat. As it was considered inadvisable to open up too many wells at one time, alternate ones only were started. As the concreting of these piers was finished, oak cribs were placed upon them to take the load of the building, as shown by Fig. 9.

As the work on the *K* piers was nearing completion and the majority of them had been blocked up, the construction of the remaining 56 piers was commenced, it being the intention to sink these piers while the righting process was under way. These were opened up as far as possible in widely scattered positions. Although quantities of water were encountered, particularly at the north end of the building, the working conditions were somewhat better than at the *K* row, and the work progressed steadily. These wells were also 7 ft. in diameter; they were placed under the contact walls and tank ends, and were in all cases carried to rock. They were arranged in four rows, starting from the east, and designated as *G, H, I,* and *J*.

At about this time the original plans were changed, owing to the fact that the railroad engineers had decided that they would prefer to have the tunnel floors above the ground-water level. To accomplish this the new plan contemplated allowing the bins to rotate about the *K* row until the angle, 16° 30′, had been reached, at which time the binhouse was to be pivoted about the *I* row of piers to an angle of 8° 30′. At this point the structure was to be again pivoted about the *H* row of piers into a vertical position.

As the drifts or tunnels were driven under the high side of the mat, 12 by 12-in. posts were set up in them on mudsills of such an area that the load imposed on the soil was about 3 tons per sq. ft. See Fig. 10. These helped to make up for that resistance against the mat which was lost as the drifts advanced. Later, as the excavation was extended so that the sinking of the wells in the *G, H, I,* and *J* rows could be proceeded with, more posts on mudsills were added, for the same reason.

The number of posts put in also enabled the settlement of the structure to be closely regulated. This was preferable to depending

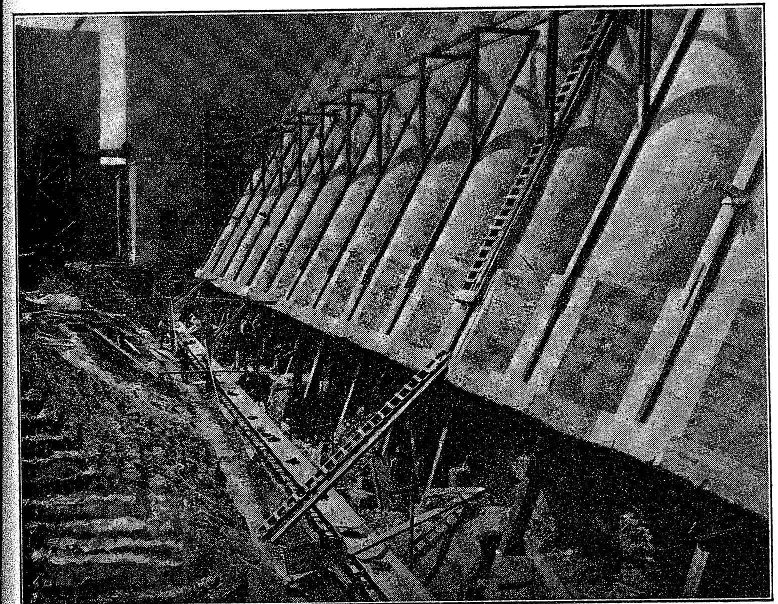


FIG. 10.—EXCAVATION TOWARD WEST FROM UNDER EAST EDGE OF MAT.

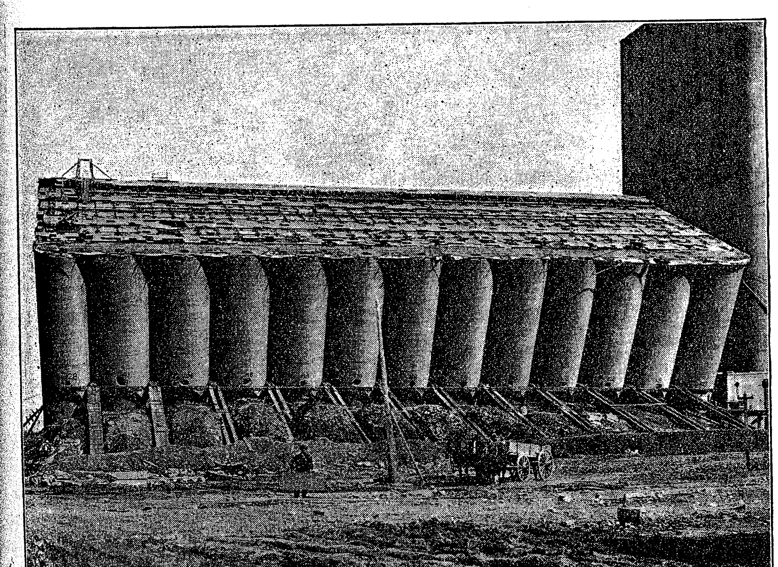


FIG. 11.—LINE OF PUSHERS AGAINST WEST SIDE OF BIN STRUCTURE, SHOWING HOLES THROUGH WHICH GRAIN WAS TAKEN OUT.

only on the clay ribs, which, on occasions, had a tendency to break off in large masses. This fracturing of the clay cores also necessitated increasing the number of posts.

The posts as well as the shoring screws, mentioned later as being placed on the G and H piers, were also necessary on account of the fact that the reinforced mat, tunnel walls, and bins were not tied together vertically in any manner. Without some such precaution the mat could have broken off, or the three parts could have slid on one another.

With the bins at rest at the angle of 26° 53′, calculations showed that the loads at the pier sites were as follows:

Those in G row	......	Uplift	
" " H "	......	11	tons per pier.
" " I "	......	227	" " "
" " J "	......	529	" " "
" " K "	......	773	" " "
Total per transverse row	....	1 540	tons.

To assist in righting the bins, twelve pushers, as shown by Fig. 11, were placed against the west side of the tanks. Each of these engaged a 12 by 12-in. waling piece placed against the side of the tanks about 45 ft. down from the top, this distance being selected so that their forces would be in a perpendicular direction to the vertical height of the tanks. The wale, in addition to resting against the thirteen tanks, was also posted into the contact walls, for it was at these points that the pushers were applied. Each pusher was composed of two 12 by 12-in. timbers, spaced 12 in. apart by using spreaders of that dimension, and tied together by plates and bolts. Each was 60 ft. long, and each had two screws, one in each of the 12 by 12-in. timbers, these screws heeling against ample timber mats. The screws were operated until the mat had reached an angle of 8° 30′. The line of action of the pushers was lowered once during the righting operation so as to maintain as nearly as possible their perpendicular direction against the sides of the tanks.

The initial righting movement, namely, that of rotating about the K row, was induced by weakening the earth partition between the drifts. Practically, a continuous daily movement of from 3 to 4 in., at the eastern edge, was maintained. When the bin floor had reached an angle of 18°, the movement was stopped for the purpose of placing the hardwood rockers at the I row of piers.

During this initial movement all the wells under the mat were successfully bottomed and concreted to the heights required for carrying on the work. As rapidly as the G and H rows were completed, cribwork was placed on each pier, with 40-ton shoring screws in contact with the mat, as shown by Fig. 12. These screws made possible a much more positive control in lowering the mat than could have been attained by the clay rib alone. The placing of the shoring screws and posts on mudsills also permitted the removal of all the clay under the mat back to the west side of the H row of wells, approximately 23 ft. from the edge of the mat. The removal of this excavation from under the eastern edge of the mat was thus carried on more economically than if the excavation had been made through the holes in the mat. In fact, as the lowering of the mat continued, a uniform depth of the clay of approximately 8 ft. from the outer edge and 4 ft. at the line back of the H row was maintained—the belt conveyor used for the disposal of the material being also lowered from time to time as the general excavation progressed.

Shortly after starting to rotate the structure about the K row, readings showed that a lateral or sliding movement to the eastward had set up. This was hard to understand, on account of the inclination of the bins to the west; also because of the compressed clay fronting the under side of the mat, and the fact that the dead load of the structure was 20 000 tons. Notwithstanding this, however, the tendency was always in evidence until the structure was pivoted on the I row of piers. To resist the movement, 12 by 12-in. timber kicking braces (Fig. 13), usually arranged in pairs, were used. These were of two lengths: short ones engaging the high edge of the mat, and long ones reaching in under the mat to notches cut in its under side, as shown by Fig. 14. In both cases they heeled against the earth embankment on the far side of the trench, where liberal timber heels had been set up. To retard the lateral movement still more, short inclined posts were set up on the K piers, their upper ends engaging notches cut in the mat. In addition, approximately 3 000 cu. yd. of earth, pressing against the bins along the west side, were removed. This total lateral movement to the west finally amounted to 3 ft. 3 in.

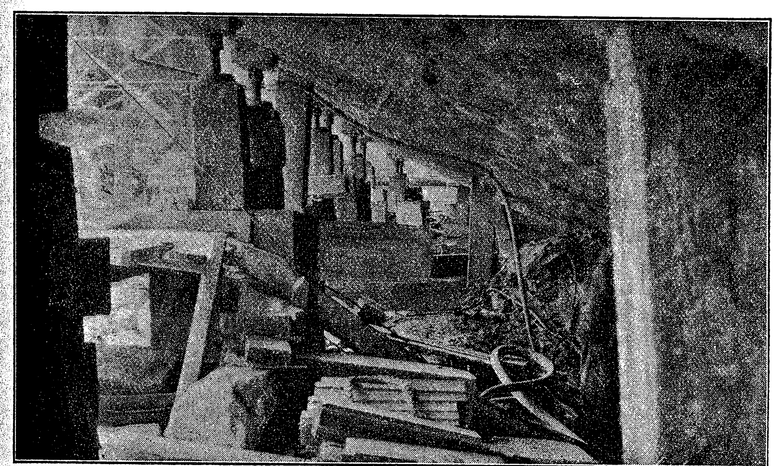


FIG. 12.—SHORING SCREWS, ON "G" ROW OF PIERS, CARRYING EDGE OF MAT.

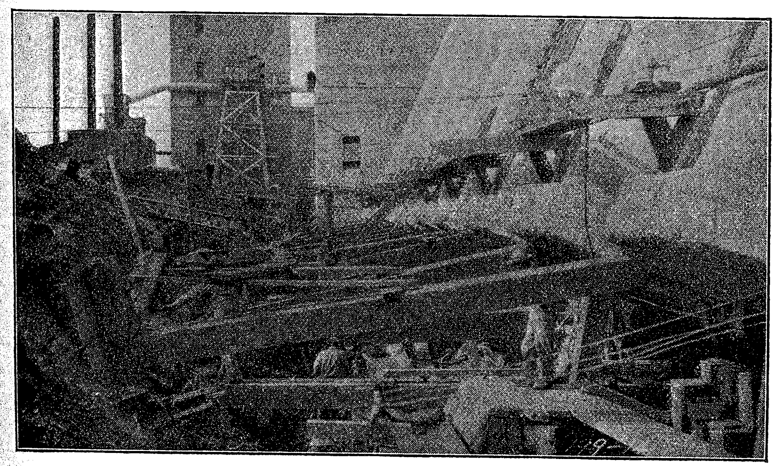


FIG. 13.—KICKING BRACES AGAINST EASTERN EDGE OF MAT AT BINHOUSE.

On August 13th the structure was brought to rest at an angle of 18° by using cribbing placed on the J row of piers. There were two of these cribs on each pier, and they were placed in a direction transverse to the longitudinal axis of the bins. They had a total area in plan of 4 by 7 ft. The rear one was 2 by 4 ft., and was removed later to give room for the shoring screws; the former one was used for following up with blocking as the structure rotated about the I and H rows of piers.

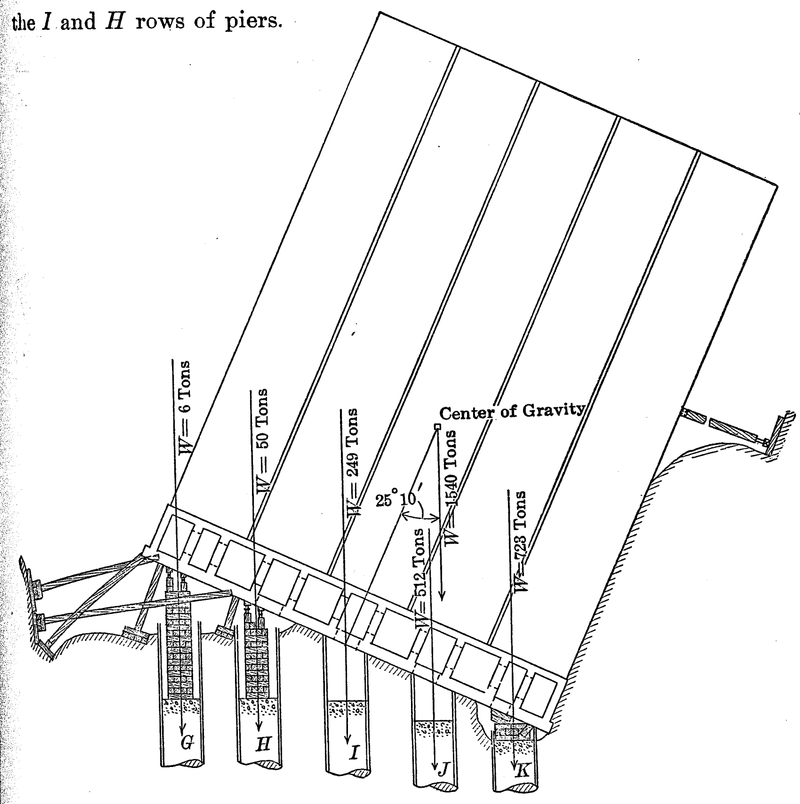
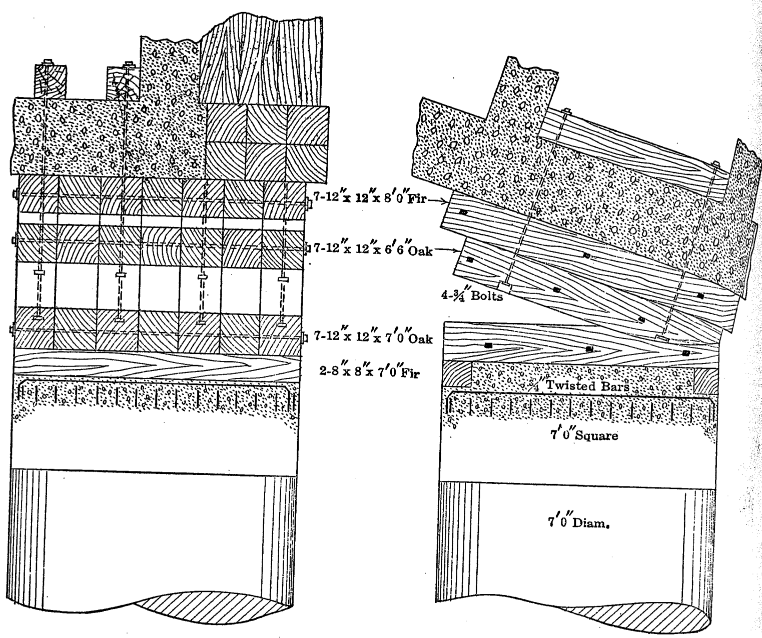


FIG. 14.

To form convenient working places, all the tops of the piers were made 7 ft. square. In preparing the piers for the rockers, as shown by Fig. 15, 8 by 8-in. timbers were embedded in the edges parallel to the longitudinal axis of the bins. The rockers consisted of two parts: the shoe and the rocker proper. The former was made by covering the top of each pier with 12 by 12-in. oak timbers, laid in the direction of the movement and strongly bolted together. The upper surface of this shoe was concaved to a radius of 30 ft., and also in the direction of the rotation.

The rockers consisted of two courses of 12 by 12-in. oak timber, seven pieces in each course, directly above the timbers in the shoe. These two courses were strongly bolted together and also to the mat. To make certain of an even bearing against the rough bottom of the mat, all uneven places were grouted. The lowest position, or heel of the rocker had previously been rounded to a radius of 15 ft. Thus the area of contact of the two surfaces was 2 ft. 6 in. by 7 ft. 0 in.



ARRANGEMENT OF TIMBERING ON "*I*" PIERS

FIG. 15.

Clearance was allowed between the rockers and the shoe, so that contact was not made until the bins reached an angle of 16° 30′. The allowance was made in order to provide for the possibility of a slight settlement during the time required to set up the rockers. When the mat had reached this angle, the center of gravity had shifted to a position 11 ft. 3 in. west of the center of the structure. At the angle

of 26° 53′, it had been 18 ft. 4 in. to the west. In the new position the figured pressures on the piers (see Fig. 16) were as follows:

G...... 80 tons.
H......193 "
I......316 "
J......423 "
K......528 "

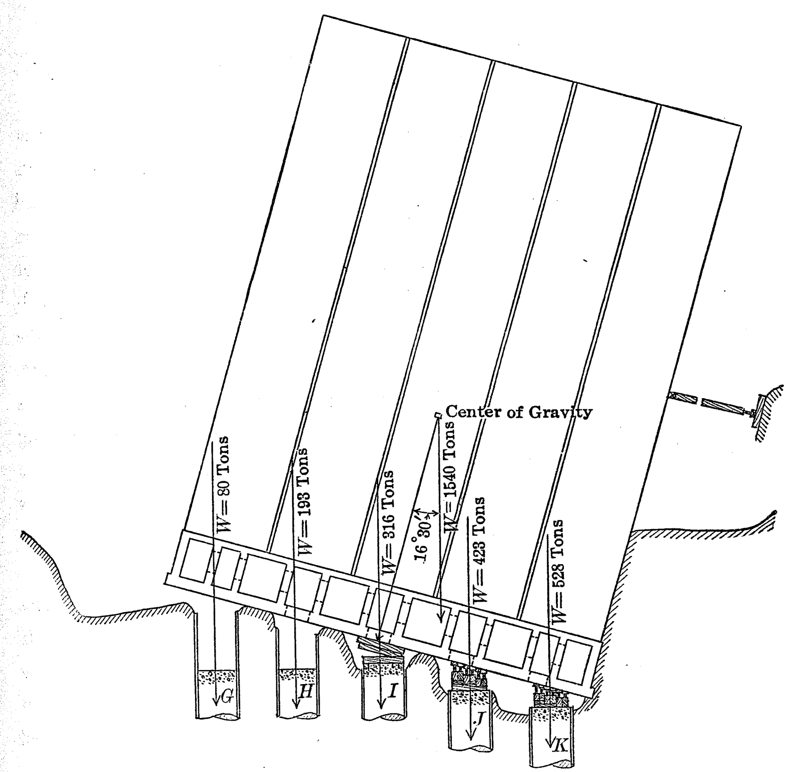


FIG. 16.

The rockers on the *I* row having been completed, the next step consisted in preparing the *K* and *J* rows for the shoring screws. Excavation to a depth to give ample room was made from the east face of the *J* piers to the west face of the *K* piers. This excavation stepped up as the *J* piers were approached, so as to maintain a practically constant head room and reduce the quantity of back-fill required later.

Between the piers of the *K* row the excavation exposed the white clay, previously mentioned as having been very much compacted. Over the clay and extending from pier to pier throughout the total length of the *K* row was laid 1 ft. of concrete. On these slabs were placed oak timbers, and on the latter four 50-ton shoring screws were set up. Twelve 50-ton screws, surrounding oak cribs 4 ft. square, were placed on the *K* and *J* piers. At the time of starting the movement there were, therefore, twenty-eight jacking units of twelve screws each and thirteen of four screws each, as shown by Fig. 17.

The 4-ft. square oak cribs were used to block up the mat as fast as the movement took place.

At the time of landing on the *I* row of piers, the mat to the east of this row had been brought in contact with the clay. From this time forward the overhang to the east of the center row was carried entirely on the clay, the shoring screws from the *G* and *H* piers having been removed to assist in jacking up at the west side. Prior to any movement of the bins, and while they were being righted, calculations were made to ascertain the weight of the structure at the different piers. These calculations were made for every 30′ of arc. From the calculations and a knowledge of the clay values, it was possible to maintain such resistances to the advance of the building as would just support the mat and prevent it from fracturing as a result of overhanging the *I* rockers. Before pivoting on the *I* row of piers, these studies also had governed the number and operation of the shoring screws and posts.

Further, the calculations of the weights at the different angles were applied to the conditions which would exist at the *I* and *H* rows of piers when used as fulcrums. These showed that the loading approached too closely to the safe value of the oak. The loading at the rocker points, therefore, was lessened by maintaining a calculated resistance of the clay under the east side. Extra screws under the low side were used to overcome this resistance.

The righting operations were continued, both by lessening the resistance of the clay under the overhanging portion of the mat and by jacking up the low side. Trenches were excavated under the mat from the *H* rows of piers eastward. These trenches extended to within about 3½ ft. of the east edge of the mat, and averaged 4 ft. in depth, except when the mat was approaching the horizontal position, when

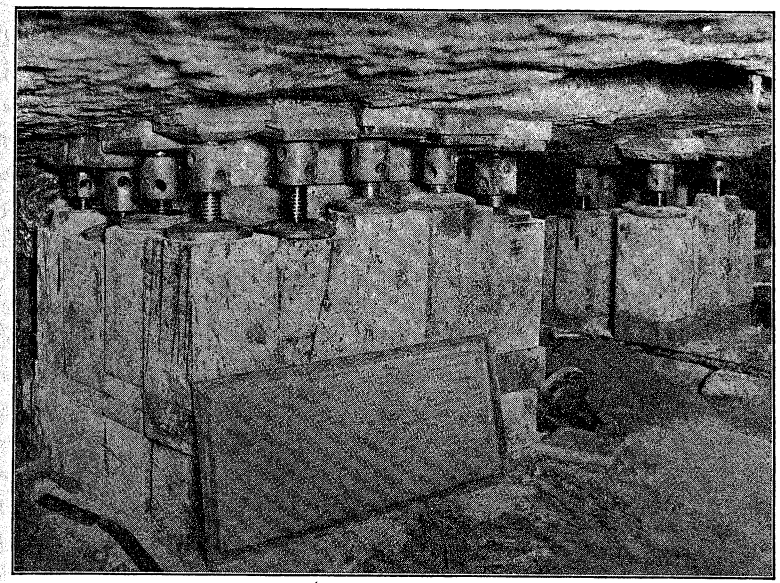


FIG. 17.—TYPICAL JACKING UNITS ON AND BETWEEN "*K*" PIERS.

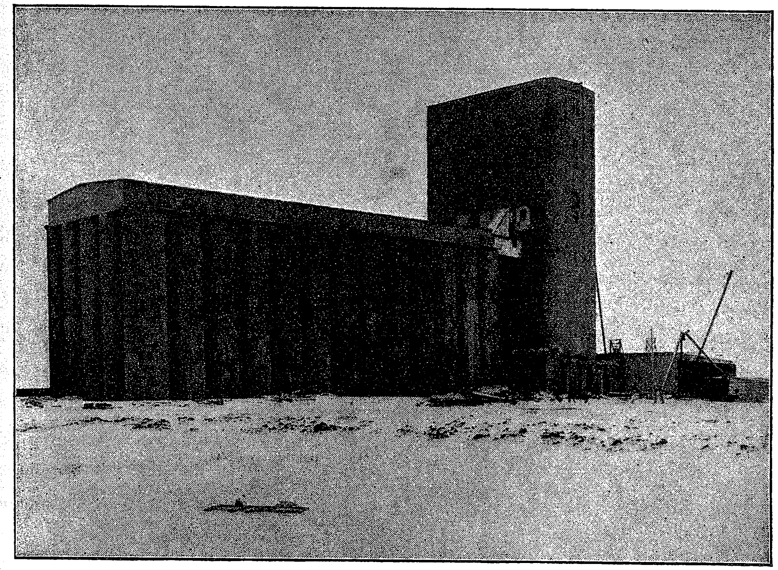


FIG. 18.—STORAGE BINS IN RIGHTED POSITION.

they were allowed to fill. The bank of clay between the H and I rows was left in place. The width at the bottom of the trenches averaged about 3 ft., the sides being sloped so that each rib between two trenches had a triangular section. Continuous weakening of the clay ribs and deepening of the trenches was maintained at a rate practically equivalent to the uplifting thrust of the shoring screws. Spoil from the trenches was handled through holes in the mat.

The jacking part of the lifting operation was done by gangs made up of three men. Each gang was supplied with a 6-ft. steel bar $1\frac{1}{8}$ in. in diameter, which, when inserted in the head of the shoring screw with three men pulling on it, was equivalent to an effort of approximately 55 tons. In the K row each gang handled eight screws, and six was the allotment for the J row. The periods of work and rest were carefully arranged, and a uniform application on the screws throughout the total length of the bins was maintained during such periods. The jacking-up process was only carried on during the day shifts. At the same time, six gangs of two men each were kept busy fleeting the screws and following up with oak blockings as progress was made. The night shift was utilized to fleet the screws, block up, and add concrete on the tops of two piers each night, and on both the J and K rows. The clay filling around the piers, to maintain the earth floor to the proper working height, was also done during the night shift.

The position of the structure was checked twice daily. In addition to levels being taken at numerous points around the structure, two verniers, fabricated on the work, were attached to the east walls inside of the north and south tanks of the G row. Each vernier consisted of an arc of a circle of 15 ft. radius, graduated to 5′ of arc. A wire with weighted lower end hung from the center of the circle. Prior to any movement of the bins, this had been set to 26° 53′. Thus the actual angle of the structure could be ascertained quickly at all times. A 240-lb. plumb-bob was also suspended from the top edge of the west face of the bins. By these devices—in addition to the angle changes—any warping of the structure, failure of the tanks to follow the mat movement, and bending of the mat could be noted.

On September 17th the angle had been reduced to 8° 30′. Jacking was discontinued while oak rockers, similar to those used on the I piers were placed on the H piers. As in the previous case, at the I

piers, allowance had been made for the creeping of the structure during the time it took to place the rockers, and they did not come to a full bearing until the angle 7° 30′ had been reached. The shoring screws were now removed from the pushers at the west side of the building and set up on the *I* piers. The grouping of screws during this, the final movement, was as follows: *K* piers, each twelve screws; *J* piers, each ten screws; and *I* piers, each eight screws. At this angle (see

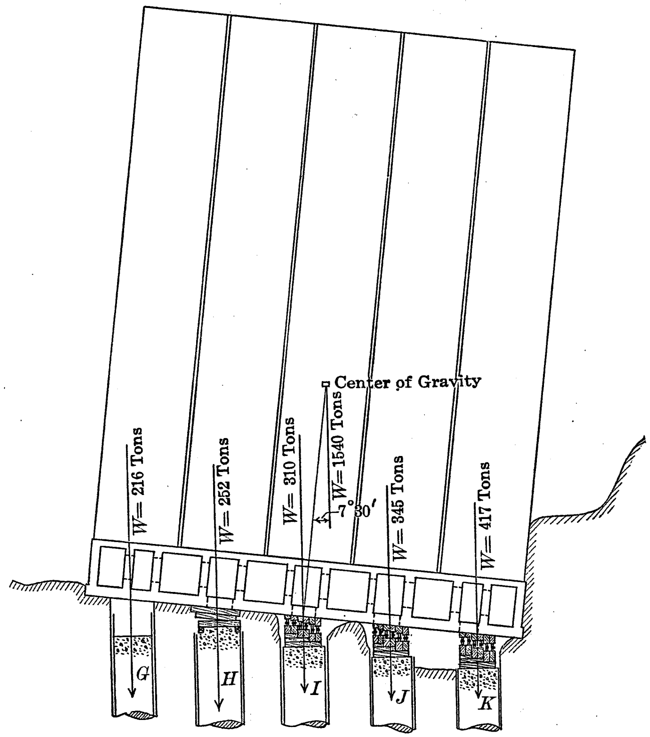
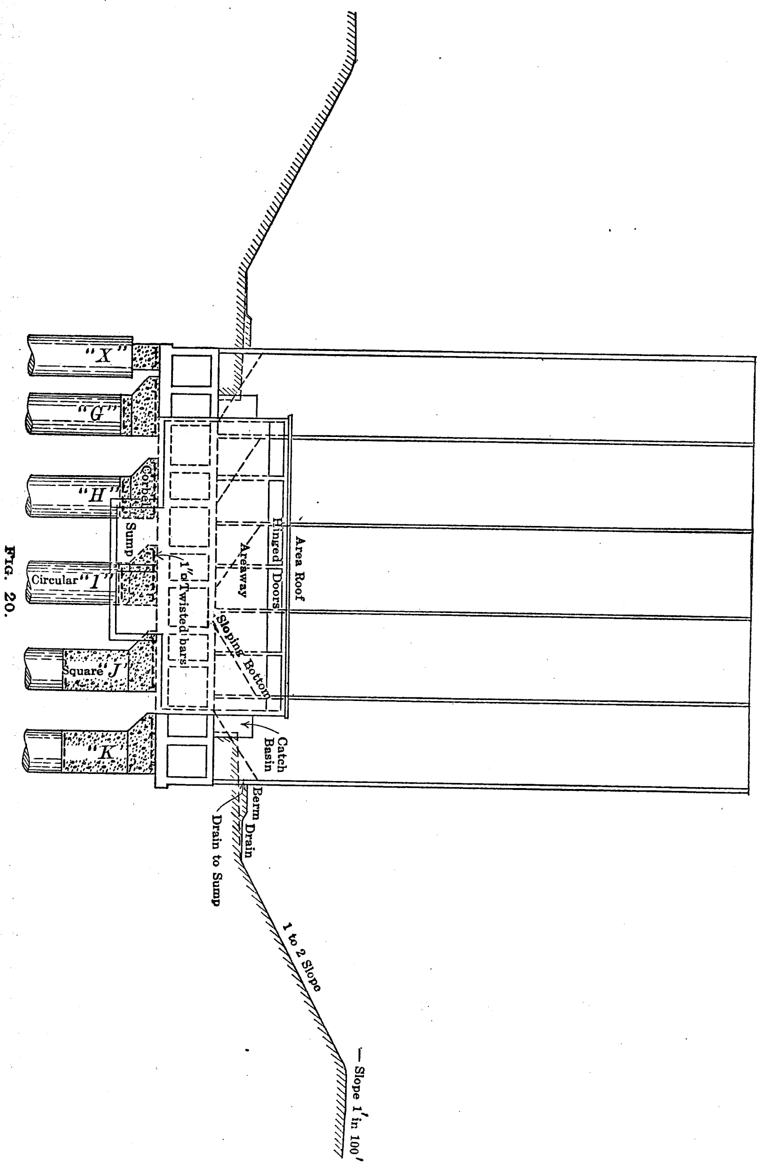


FIG. 19.

Fig. 19) the pier loading was as follows: *G*, 216 tons; *H*, 252 tons; *I*, 310 tons; *J*, 345 tons; and *K*, 417 tons. The center of gravity, in the meantime, had shifted until at this angle it was only 3 ft. 3 in. west of the center line of the structure. During this last operation it was found advisable to assist the screws on the *I* piers by using oak wedges, 4 ft. long, 6 in. wide, and 3 in. thick at the butt end; these were driven between the shoe and the rocker on these piers. This was made necessary on account of the tendency of the mat to sag.

FIG. 20.



An average rate of 4 in. in 10 hours, measured on the center line of the *K* piers, was maintained throughout the jacking portion of the righting operation. The binhouse was back in its proper vertical position on October 17th, 2 days behind the estimated time. After the building had been righted, the screws and blocking were removed from piers in scattered locations. In concreting up to the under side of the mat, these piers were corbeled out, so as to distribute the pressure, as shown by Fig. 20. The building as it stands to-day is practically 14 ft. below its original position, the total lift having been a trifle more than 12 ft.

To meet the new elevation of the tunnels, excavation was made in the workhouse, and the grain boots were lowered to receive the discharge from the tunnel belts. Also, to provide for the overhang of the bins on the east side, caused by the lateral movement, seven piers were constructed with corbeled tops to carry the outer points of the tanks.

The inclination of the bins to the north, amounting to 4 ft. in their total length, was allowed to remain. This does not affect the stability of the structure, nor the cost of handling the grain, so that any additional expense to get the structure exactly level north and south would have been unwarranted.

At the north end of the structure, an areaway, 6 ft. wide and 52 ft. long, was built to a height of 15 ft. above the tank bottoms. This was roofed over with concrete, 2 ft. 6 in. above the top of the walls, the open space being left open for the ventilation of the tunnels. At the bottom of the areaway was built a sump. This extends 7 ft. below the mat, and is 6 ft. wide and 21 ft. long. Here were installed two No. 4, motor-driven, submerged type, centrifugal pumps, having a rated capacity of 470 gal. per min. As shown by Fig. 20, no attempt was made to back-fill around the tanks to the prairie level. Leaving the depression as shown precluded the necessity of water-proofing the tanks, which would have been necessary to protect the grain had the soil been allowed to come in contact with the thin walls. It will be noted that a berm 8 ft. wide was left around the structure. This slopes away from the bins, and forms a drain for carrying the surface water to catch-basins placed just outside of the north areaway, whence it is piped to the sump.

DISCUSSION

DAVID GUTMAN,* M. AM. SOC. C. E.—The speaker once had an experience with the sinking of the foundation of a building which, although of a different nature from the grain elevator under discussion, had one point in common, in that the structure acted as a monolith. Mr. Gutman.

About 4 years ago, he was designing engineer for the architect on the Taft Hotel, at New Haven, Conn., a fourteen-story structure with two basements. The framework was of steel, and the floor-slabs were of the long-span type formed of concrete beams, 16 in. on centers, with a tile filler between. The foundations consisted of cast bases resting on reinforced concrete piers, the piers resting on sand. In the boiler-room the tops of these piers were about 4 ft. below the water line. The piers were placed by draining the water to a sump and pumping day and night. The basement was then water-proofed by placing felt water-proofing on a concrete mattress. The water-proofing was in turn held down by a concrete floor, the reinforcement being inverted, as the load—that is, the water pressure—was from below.

Early one morning, the speaker, who was in New York, received an urgent telephone call to visit the job at once, and on his arrival found that one of the concrete piers had dropped more than 2 ft., taking the cast base with it and leaving the column suspended in mid-air. The column itself had sunk about 2½ in.

The column was of the I-type, and had a girder running into each flange, the depth of the girders varying from 15 to 20 in. on the different floors. It was evident that the column no longer acted as such, but simply as a link between the two girders, making them one continuous girder of 32 ft. span, instead of two girders of 16 ft. span each. No wind bracing had been provided, other than a very stiff girder connection to the columns. One of the features of this connection was a 6-in. outstanding leg for the angle-seat and top-lug, having four rivets instead of the usual two. It was this stiff connection, as well as the excellent floor-slab (from 7 to 9 in. deep), which made the continuous action possible and prevented a very serious collapse.

The cause of the sinking of the foundation—a concrete pier 10 by 10 ft.—was as follows: The felt water-proofing had been pierced in some way, and this allowed a gradual seepage of water which accumulated in the cellar. In order to get rid of this water the pump was again set going, but, through negligence, the damage was not repaired at once, the work being delayed almost a month. Of course, some sand was pumped in with the water, and gradually the pier was undermined.

* Mount Vernon, N. Y.

Mr. Gutman.

Two heavy plate girders were placed on opposite sides of the column, spanning to the two good piers on each side. These girders were then riveted to the column, and jacks were placed under the ends. By these jacks, the column was pushed up about ¾ in. It was not considered wise to raise it the whole amount of the settlement—2½ in.—as this would have cracked the plaster on the upper floors, most of which had already been placed. The piers on each side showed no settlement whatsoever, either before or after the jacking.

Mr. Phillips.

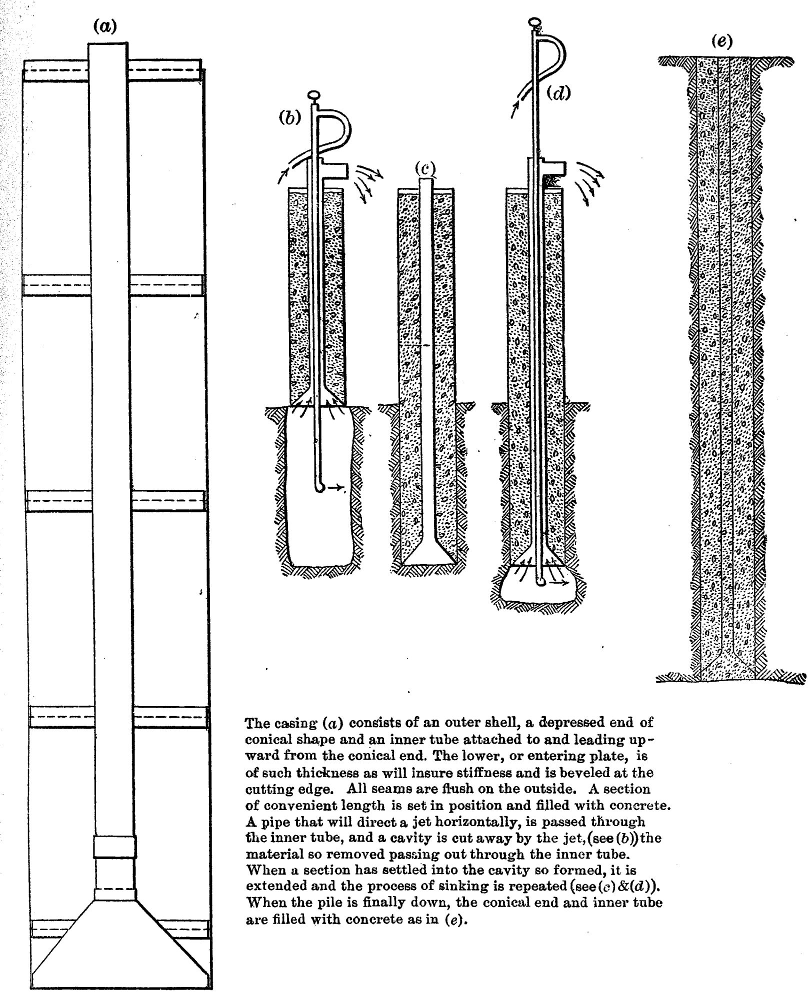
W. R. Phillips,* M. Am. Soc. C. E. (by letter).—A brief description of some work brought about through the failure of the foundations at the Union Meat Company's plant may be of interest in connection with the discussion of this paper. The cases were to some extent similar, although the manner of sinking the new foundations was different. Unlike the work at Winnipeg, this was handled entirely from the surface, and was independent of underground conditions, such as water, depth, or material, so long as the latter could be cut by the jet.

The plant of the Union Meat Company is housed in a group of buildings suited to the packing business, and is on the south shore of Oregon Slough, Columbia River, at North Portland, Ore. The principal buildings of the group are the main packing house and the tank house. The former is 122 by 166 ft., and, in its highest part, has seven floors. The latter is 33 by 52 ft., and has four floors.

As designed originally the buildings were to rest on concrete foundations of considerable depth, supported by wooden piles. The piles were properly driven and, after the foundations were built, the spaces between and around the walls were filled with sand dredged from the river. In preparation for the work, wash borings had been made to a depth of 116 ft. below the surface of the ground, where cemented gravel was found. The borings produced nothing to indicate that the ground would not remain stable under the load to which it was to be subjected, and, in this case at least, may be considered as an illustration of unreliability in a test of this class, as the condition on which the ground depended for its stability was not such as would be readily discovered by wash borings. It was not until the concrete piles for the new foundation were being sunk that the clue was obtained whereby the condition that might be responsible for the subsidence was discovered. During that process a horizontal stratum of dense material about 6 ft. thick, was found at a depth of about 65 ft. below the surface, and, as it happened, just beneath the ends of the wooden piles, which aroused suspicion that the water beneath it was sustaining the ground, and that the water was being slowly displaced by the added load. The correctness of this theory was seemingly demonstrated by sinking

* Portland, Ore.

Mr. Phillips.



The casing (a) consists of an outer shell, a depressed end of conical shape and an inner tube attached to and leading upward from the conical end. The lower, or entering plate, is of such thickness as will insure stiffness and is beveled at the cutting edge. All seams are flush on the outside. A section of convenient length is set in position and filled with concrete. A pipe that will direct a jet horizontally, is passed through the inner tube, and a cavity is cut away by the jet, (see (b)) the material so removed passing out through the inner tube. When a section has settled into the cavity so formed, it is extended and the process of sinking is repeated (see (c) & (d)). When the pile is finally down, the conical end and inner tube are filled with concrete as in (e).

Fig. 21.

Mr. Phillips. a pipe through the dense material. In this pipe water was found to stand at a height of 10 ft. above that of the adjacent slough.

Not until sand was dredged in to fill around the foundation was there evidence of settlement, but with that to start it, it was soon demonstrated that the foundation, as well as the ground around it, was subsiding. A consideration of the conditions led to the conclusion that nothing above the cemented gravel which had been found when making borings could be relied on to sustain the load.

The method adopted for supporting the buildings, and which resulted in finally cutting away and abandoning the original foundations, was to sink concrete piles, in pairs on each side of the old foundation walls, to a bearing on the cemented gravel, and to support the walls and piers on reinforced concrete beams carried by the piles.

The pile as used consists of an outer shell of steel plate attached to an entering piece called the "point", and an inner tube also attached to the point, the space between the outer shell and the inner tube being filled with concrete. The "point" may be considered as a continuous circular gouge with a cutting edge. On the inside, from the cutting edge upward, there is a short conical piece which is also connected to the inner tube and forms an inverted funnel of which the inner tube is the spout. The work connected with sinking the pile is done through the inner tube.

In its application, the "point" and a section of shell of convenient length is set in position, with a guide-frame around it to start it straight in its descent, the annular space inside the shell being filled with concrete. A pipe, which may be moved vertically or rotated at will, with a nozzle which will throw a jet of water horizontally, is inserted in the tube and connected with a pressure system by a hose or other device. Manipulation of the jet bores a hole of sufficient size into which the pile may settle, the material thus removed passing upward and out with the water, by way of the inner tube. When this first section of the pile has reached a level in sinking that makes it necessary to lengthen it, courses are added to the shell, the tube is extended, concrete is placed, and sinking is resumed. The process of extension is repeated until the pile has reached the required depth, and then the space beneath is washed thoroughly and concrete is placed there as well as in the tube, making a column of uniform strength throughout its length, and in which the concrete can all have been properly handled and inspected.

Fig. 22 shows two piles being sunk. A man on the highest staging is manipulating the jet pipe with a wrench. The pile to the left has been sunk to a point where it requires to be extended. The inner tube has been lengthened, and courses are to be added to the outer shell. This will then be filled with concrete and the sinking will be resumed.

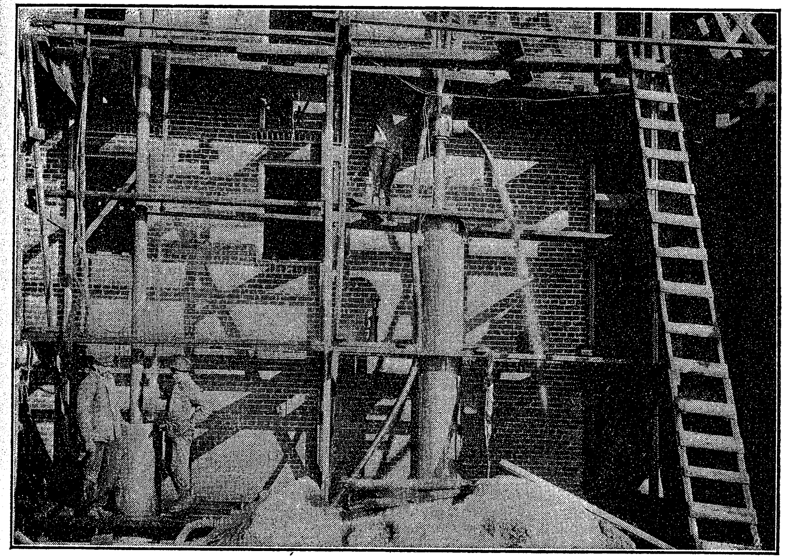


Fig. 22.—Method of Sinking Piles, Union Meat Company's Plant, North Portland, Ore.

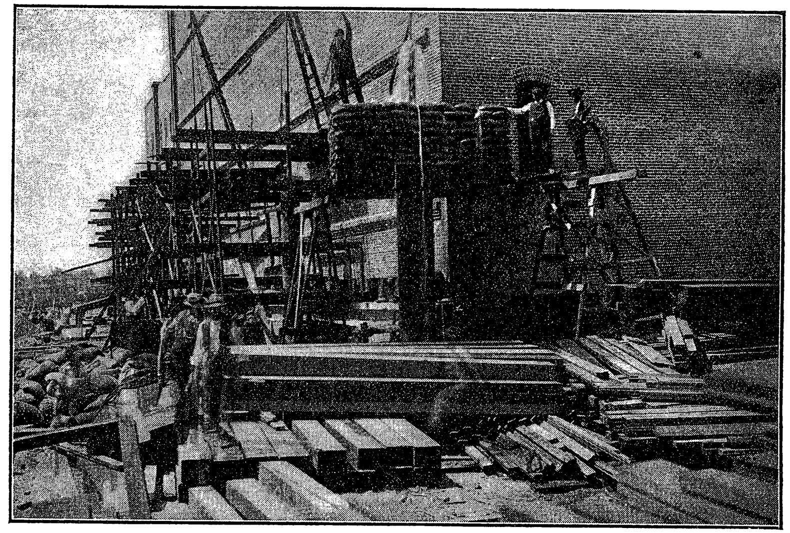


Fig. 23.—Method of Forcing a Pile by Loading.

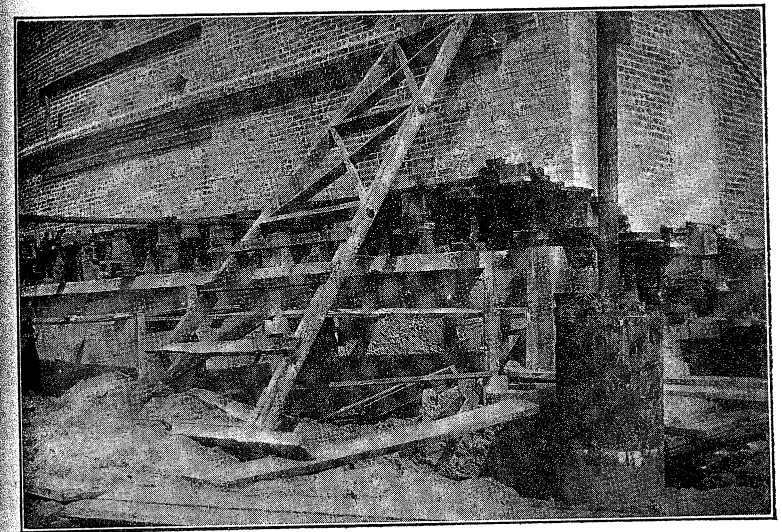


Fig. 24.—Same Pile as Shown in Fig. 23, After Sinking was Completed.

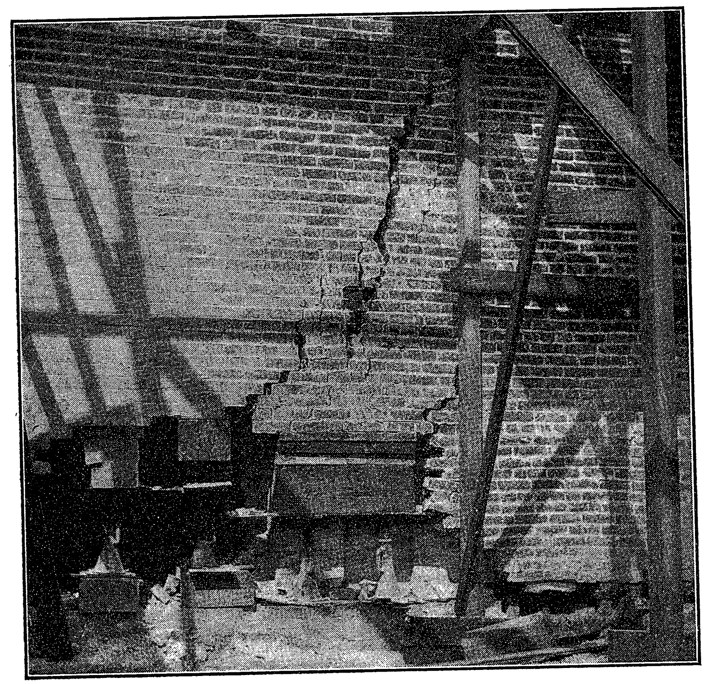


Fig. 25.—Crack in Wall of Building Caused by Tendency of Blocking to Roll.

Fig. 23 shows a pile which has reached a depth where skin friction offers too much resistance to be overcome by the weight of the pile alone, or even by a process which was called the "blow." This consisted in shutting off the water where it is seen escaping above and thus forcing it to find an outlet around the outside of the pile, thereby lessening the skin friction. When the "blow" was not effective, the pile, if on the outside of the building, was loaded as shown, and, if inside, was forced down with jack-screws. Mr. Phillips.

Reinforcement may be used in such portions of the pile as may require it, but it was not necessary in the work described. In this work, 153 piles, ranging in diameter from 22½ to 30½ in. and in length from 108 to 112 ft., were sunk.

For the support of the walls of the main packing house, the piles were put down in pairs, each pair supporting a reinforced concrete beam which was to carry its share of the wall. When the piles required for the purpose were completed, and in advance of the completion of those intended to support the columns inside the building, the beams they were to support were poured, and in due time the weight of the walls was brought to a bearing on them.

Fig. 24 shows the same pile as Fig. 23, the sinking being completed. The lower end of this pile rests on cemented gravel, 116 ft. below the surface. The blocking and jack-screws show the method of holding up the brick walls while the foundation was sinking at the rate of 0.1 ft. per day. At the corner, inside the building, the mate of this pile has been sunk. A hole will be cut through the foundation, below the surface, and a reinforced concrete beam will be built on the two piles.

Fig. 25 shows one of several cracks in the walls of the building caused by the tendency of the blocking to roll. Such fractures had to be closed up after the building had been brought to a state of rest. After piers of brick set in cement mortar had been substituted for the wooden blocks and wedges, there was no further tendency to roll.

During the progress of the work this building had been sinking constantly, but at rates that varied according to the distance from one corner of the building on the end nearest the slough, which had remained stationary. When, finally, the walls were brought to rest on the new foundation, the subsidence at some points amounted to fully 2 ft.

The columns supporting the floors of the three departments of the main packing house were in three rows in each department. To carry these, two intermediate rows of piles were sunk in line with the original piers, and reinforced concrete beams resting on the intermediate piles carried the three columns in alignment with them.

The tank house, being a lighter structure, was handled in a different way. The concrete piles for it were set away from the old founda-

Mr. Phillips. tion, and on these, continuous beams were poured. The building was moved over to the new foundation.

The work of erecting the buildings had progressed simultaneously with that of the new foundations. That it might be maintained in as nearly a level condition as possible, the walls and columns had been carried on jacks and oak wedges which were kept constantly adjusted. In spite of the care used to prevent it, the walls were badly cracked, and required considerable repairs.

There is no intention to convey the impression that there were no difficulties in sinking the piles, but such difficulties were readily overcome. It was seldom that a pile would follow the jetting as it proceeded. Instead, a cavity of some depth usually had to be washed out beneath, into which the pile would sink in due time. In general, the first 50 ft. or so of sinking was comparatively easy; but beyond that depth, in most cases, more or less urging had to be resorted to. If, however, larger piles had been necessary, or if a greater penetration had been required, or both, the work could still have been accomplished without undue difficulties.

Mr. Goodrich. E. P. Goodrich,* M. Am. Soc. C. E.—This paper appeals to the speaker more strongly with reference to incidental and exterior phenomena than with reference to the ingenious and eminently satisfactory method used in righting the elevator, which is the primary reason for the paper.

The first item of particular interest to the speaker is the demonstration that a structure of the size of this grain elevator, when built of reinforced concrete in accordance with the proper design, becomes a monolithic structure capable of being subjected to extraordinary stresses without seriously damaging the building. The settlement and righting of this elevator are almost identical with an experience concerning a grain elevator in Algiers, erected and righted by French engineers.

The moral of this story is an indirect demonstration of the need of caring for reverse moments in all bottoms, and at the points of intersection of bottoms and columns, together with the need for care in the design of columns to resist bending where need is shown. So long as reinforced concrete members were designed after the fashion of those made of timber or steel, difficulty was sure to arise, but, with the wider adoption of the cantilever, difficulties have disappeared and better structures been secured. The speaker has been interested in noting the widening use of the cantilever, even in steel and timber work, and believes that members thus designed are more efficient than almost any other device for the same purpose.

Reverting to the fact of the monolithic nature of large reinforced concrete structures, many instances may be cited of caissons designed

* New York City.

for use as foundations for bridges, docks, etc., which have been handled, lifted on one side, tilted, and manipulated in almost any way found necessary without great difficulty. Mr. Goodrich.

The second point of interest refers to the earth pressure phenomena described. The wave that was thrust up, perhaps from some deep stratum, can be made the source of information with reference to earth pressure matters, which may prove of value through an analysis of its size, rate of formation, etc., if such data can be made available. For a long time the speaker has been advocating the making of careful explorations of sub-strata under large buildings, going down many feet below the customary foundation depth. Demonstrations of this need, in addition to that afforded by the Canadian Pacific grain elevator, were also mentioned by Robert B. Stanton, M. Am. Soc. C. E., in his discussion* on earth pressure formation in connection with a slide which took place on the Canadian Pacific Railroad within his observation.

The upheaval of the bottom of the Panama Canal is a gigantic demonstration of the same fact. On a much smaller scale, the upheaval of a steam shovel excavation in Cleveland, which occurred several years ago during the cutting through of a street, is pertinent.

It is the speaker's belief that properly made explorations and physical tests of the soil secured will make it possible to prophesy the possibilities of just such failures as occurred in connection with this grain elevator. Where such possibilities are indicated, obviously, engineers can take all needed precautions to prevent their occurrence. This belief has been applied by the speaker in several foundation investigations, and it is equally applicable to the analysis of soil conditions at Panama or Winnipeg. This paper is thus believed to be of wider interest than that covered in its main description of the mechanical methods used in combating the particular difficulty here encountered.

* *Transactions*, Am. Soc. C. E., Vol. LIII, p. 307.

TRANSCONA ELEVATOR FAILURE: EYE-WITNESS ACCOUNT

by

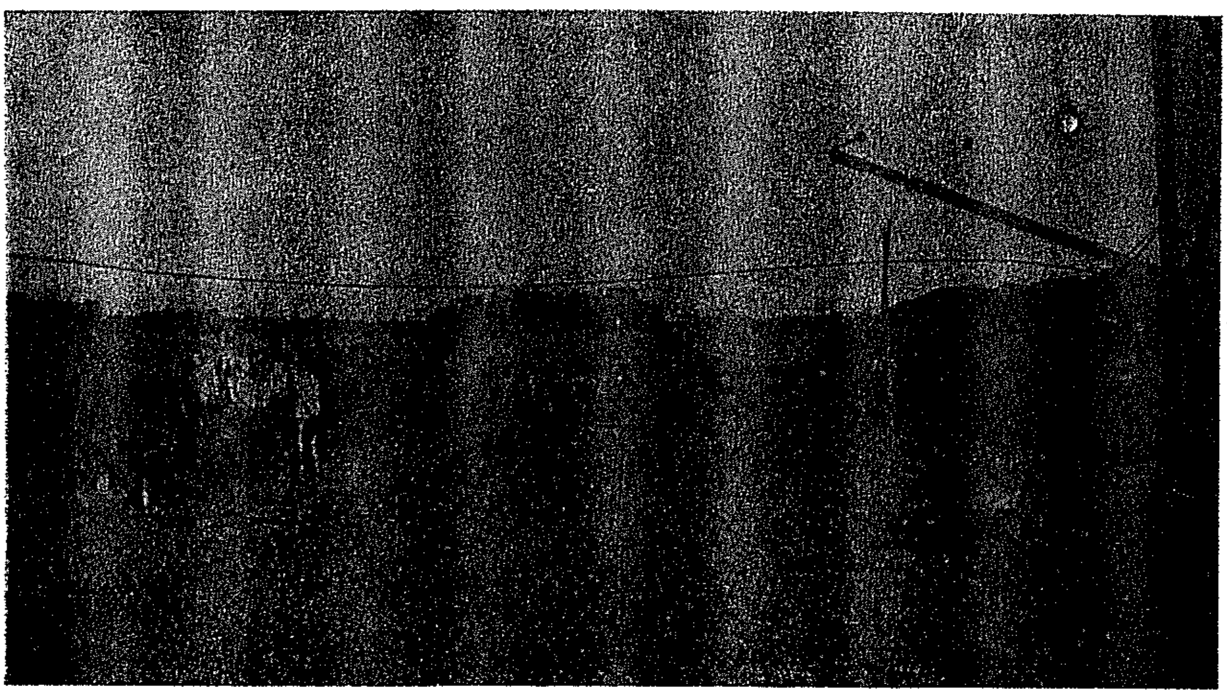
L. Scott White, O.B.E.

The Grain Elevator referred to in the following article was one of the most important structures built by the Canadian Pacific Railway Company in their extensive railroad yards at North Transcona. The yards covered several square miles, and were built on partly farmed prairie land. The ground was relatively flat for miles around.

Excavation for the foundations of the elevator was in open cut about 12 feet in depth (see Fig. 1). Bearing tests were made by loading a plate laid upon a prepared smooth surface; the test loading was applied from a wooden gantry erected for the purpose. To the best of the Writer's memory no borings were taken.

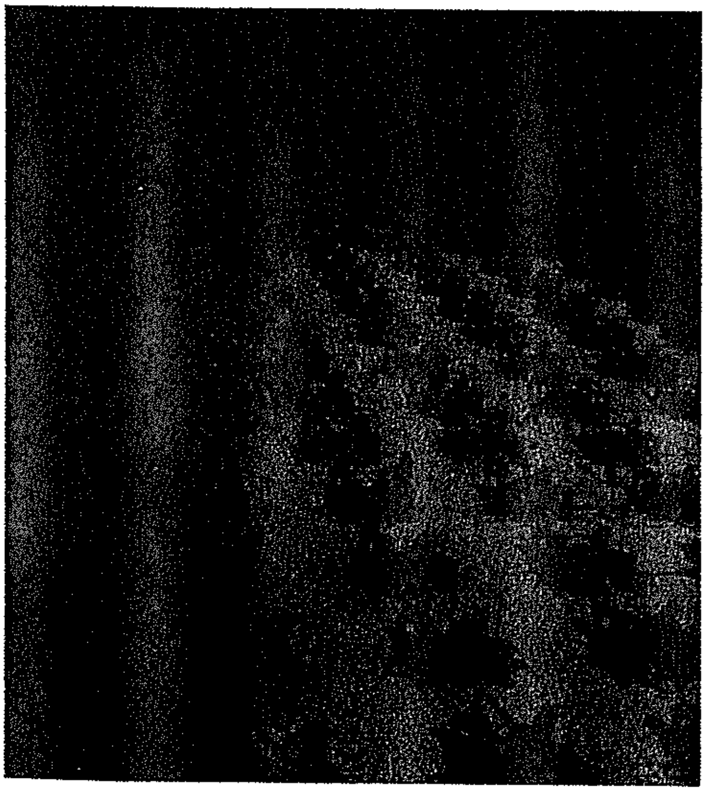
The tests appeared to satisfy the requirements of the engineers, who assumed that the "blue gumbo" had similar characteristics and depth to that on which many heavy structures had been founded in the vicinity of Winnipeg, 7 miles to the west.

Fig. 1



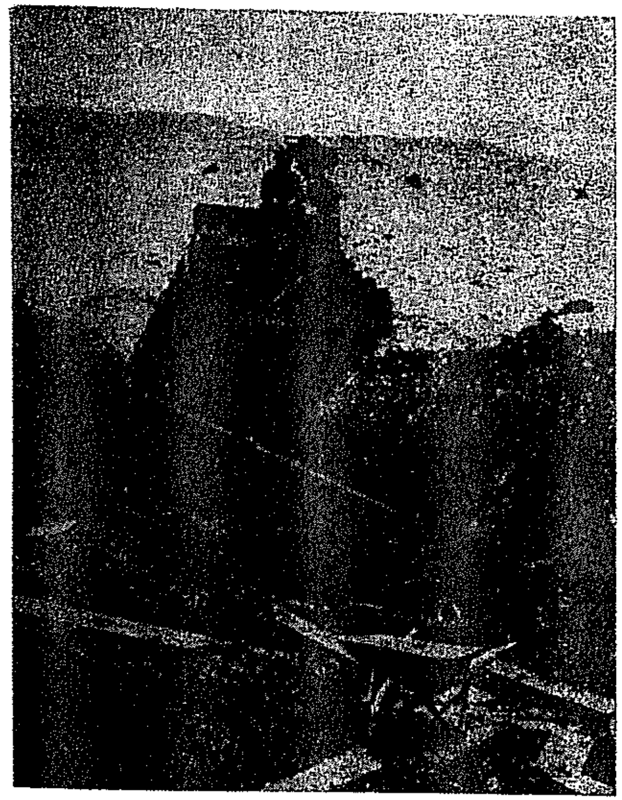
Site on commencement of excavations

Fig. 2



Circular bins

Fig. 3



Subsidence caused by thaw

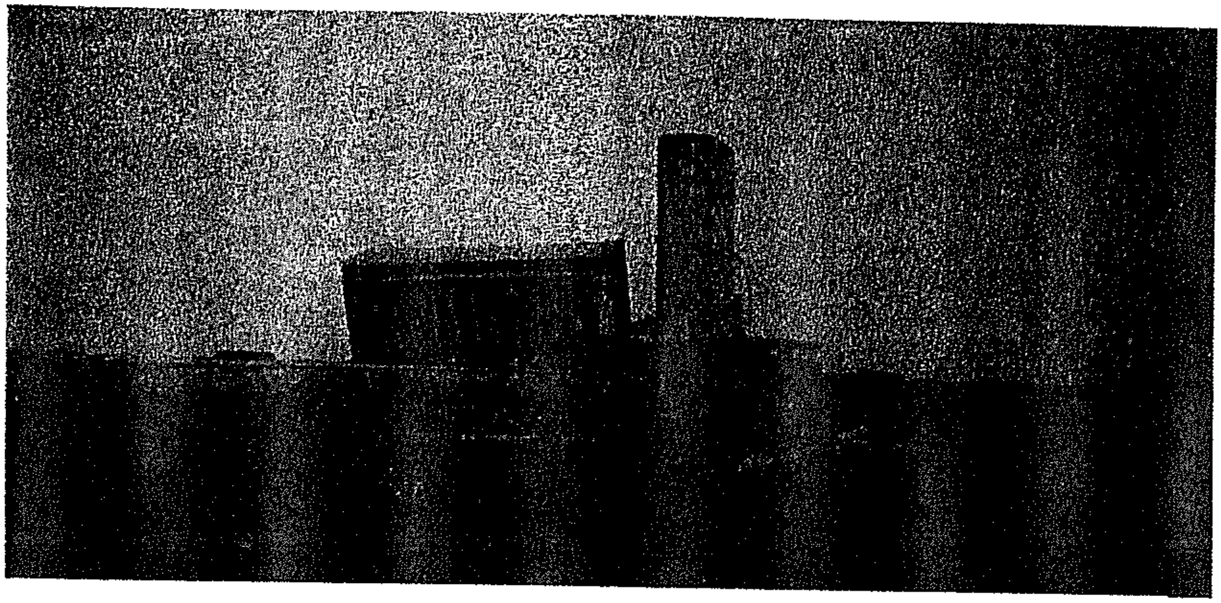
Fig. 4

60-foot timber piles forming staging for track

The construction of the elevator proceeded at a rapid rate during the autumn and winter of 1912, the circular bins being raised at the rate of 3 feet per day (see Fig. 2).

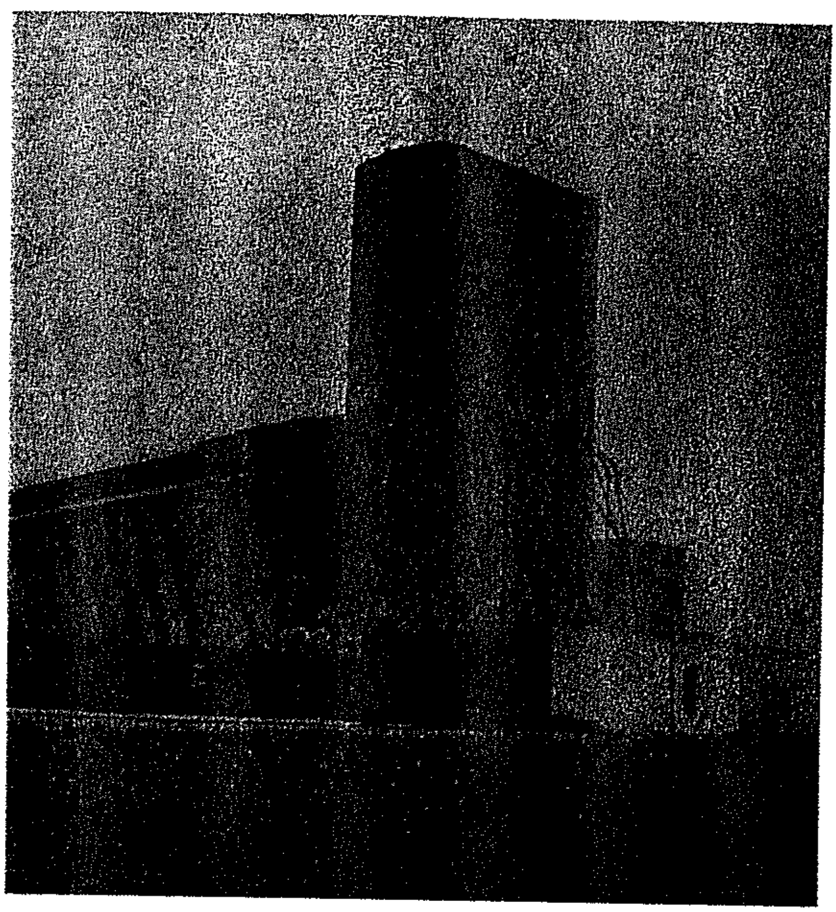
In the spring of 1913, when the thaw set in after heavy winter snows, a great deal of trouble was caused throughout the yards by the movement of the top surface of the clay under the pressure of deep ballast-fills which carried the tracks. (Fig. 3.) In one case high-level tracks, which were laid on a 30-foot fill, subsided several feet, throwing up great rolls of ground to the sides and breaking up massive drainage culverts constructed of 12-inch-by-12-inch timbers. This subsidence was remedied by driving hundreds of 60-foot timber piles through the ballast to form a staging on which the tracks were relaid. (See Fig. 4.)

Fig. 5



Movement of elevator during first half-hour

Fig. 6



Close-up view showing extent of movement

This was not the end of troubles, for on the 18th October, 1913, when the engineering staff of the C.P.R. were having their lunch at a camp about a mile distant from the elevator, there were excited cries that the elevator was collapsing. Everyone rushed out to witness the phenomenon not believing it possible, but, sure enough, the bin structure had already taken a considerable tilt to the west and was still moving. The Writer, who possessed the only camera in the party, hurried over to the scene taking photographs *en route*. Figs 5 and 6 show the extent of the movement within the first half an hour. It will be noted that the cupola is still in place. As the structure tilted to the west, the earth on that side bulged up forming a cushion which slowed down the movement. (Fig. 7.) On the east side a wide gap was left in the ground to the depth of the raft foundation.

The movement of the bin structure throughout was gradual and barely susceptible to the eye, but a considerable amount of commotion was caused by the connecting bridges carrying the conveyor belts breaking adrift and crashing to the ground.

The main concern of everyone at the time was whether the work-house could stand the disturbance. Check levels were therefore taken and it was found with considerable relief to to be standing firm.

Movement continued at a diminishing rate for the rest of the day. In the night the cupola structure over the bins, which housed the conveyor belting, suddenly collapsed and fell to the ground. This reduced the load and there was subsequently little further movement.

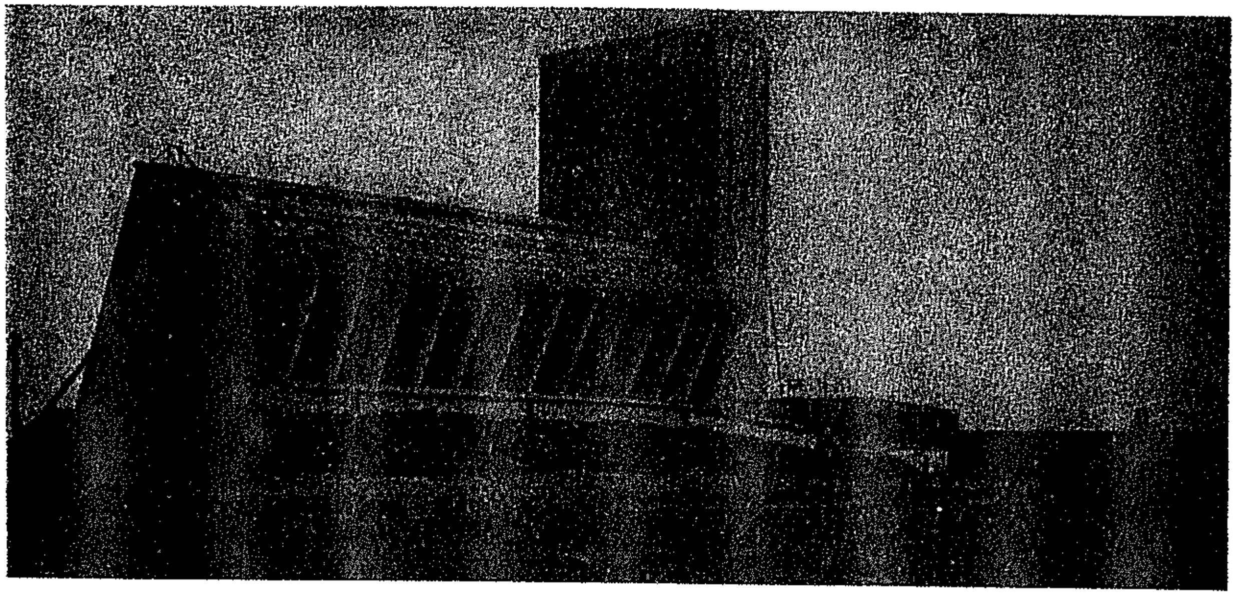
Naturally a great strain was put on the comparatively thin walls of the storage bins but they were so well constructed that hardly any cracks appeared in them.

An account of how the grain was discharged (Fig. 8) and how the bin structure was underpinned and jacked-up from new pier-foundations taken to solid rock is contained in Allaire's Paper "The Failure and Righting of a Million-Bushel Grain Elevator" reference to which is made in Professor Peck's article (see Figs 9 and 10).

Fig. 7

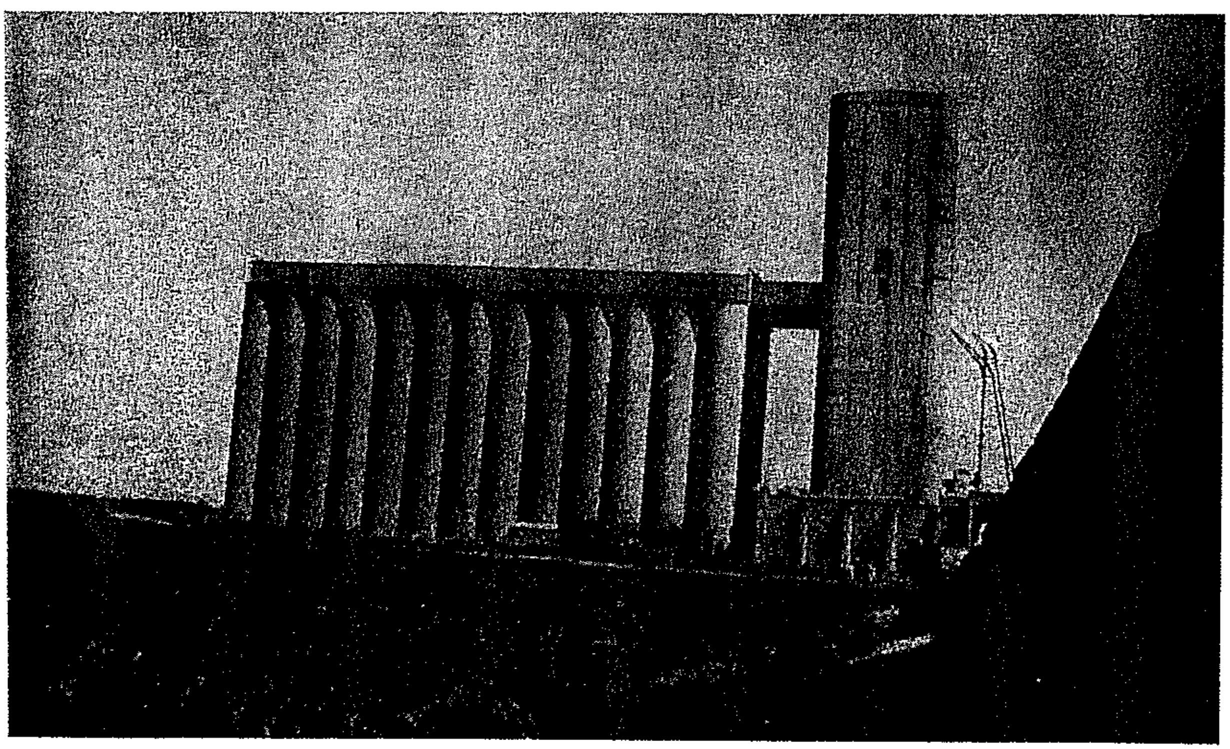
Cushion of earth caused by tilting structure

Fig. 8



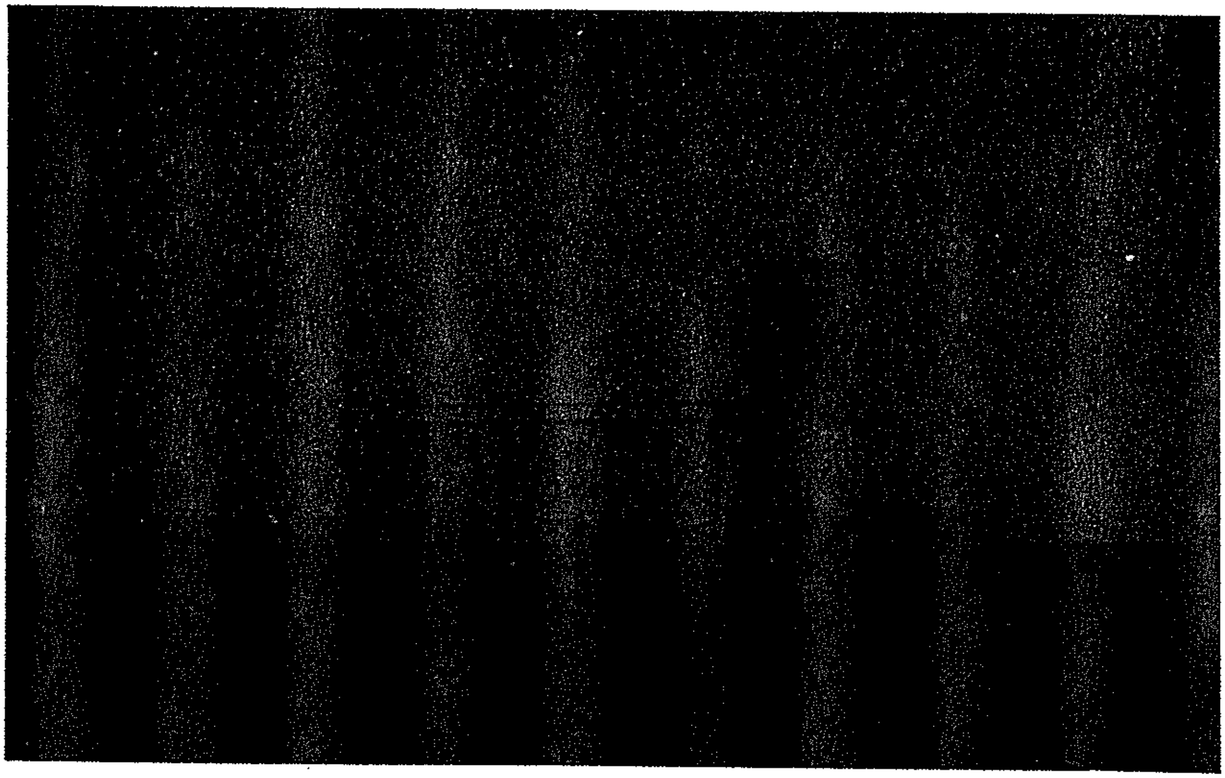
Elevator being discharged and underpinned

Fig. 9



Elevator before subsidence

Fig. 10



View of elevator after righting : note tilt

Soil Mechanics as a special science had hardly begun at that time. If as much had been known then as is known now about shear strength and behaviour of soils, adequate borings would have been taken and tests made and these troubles would have been avoided. We owe more to the development of this science than is generally realized.

THE BEARING-CAPACITY FAILURE OF THE TRANSCONA ELEVATOR

by

R. B. PECK AND F. G. BRYANT

SYNOPSIS

In 1913, a grain elevator near Winnipeg, Canada, experienced a bearing-capacity failure during its first loading. The foundation consisted of a large rectangular raft resting on a plastic clay deposit. The load at failure is known. This article presents the results of tests made to determine the properties of the clay, and demonstrates that the failure occurred at a pressure corresponding to the computed ultimate bearing-capacity of the clay.

The article is accompanied by a supplementary note by Mr L. Scott White—an eye-witness of the incident.

En 1913 la rupture d'un silo près de Winnipeg, Canada, a eu lieu dont la cause était l'insuffisance de la force portante. Le silo était fondé sur un radier supporté par une couche d'argile plastique La charge au moment de la rupture est connue.

Les qualités de l'argile sont indiquées par les resultats presentés ci-dessous des essais de sol sur des énchantillons intacts.

Ces résultats indiquent que la rupture a eu lieu sous une charge égale à la force portante limite calculée de l'argile.

L'article est accompagné d'une note supplémentaire par M. L. Scott White, qui était témoin à l'incident.

INTRODUCTION

In 1911, the Canadian Pacific Railway Company began construction of a million-bushel grain elevator at North Transcona, 7 miles north-east of Winnipeg, Manitoba, Canada. The elevator, together with one of the largest railroad gravity yards in the world, was to provide relief for the Winnipeg Yards during the months of peak grain-shipment.

The structure consisted of a reinforced-concrete work-house, and an adjoining bin-house. The work-house was 70 feet by 96 feet in plan, and 180 feet in height, with a raft foundation at a depth of about 12 feet. The bin-house contained five rows of 13 bins, 92 feet in height and 14 feet in diameter, resting upon a concrete framework supported by a reinforced-concrete raft. The raft had a width $B = 77$ feet, and a length $L = 195$ feet, and was established at a depth $D = 12$ feet below the level of the surrounding ground.

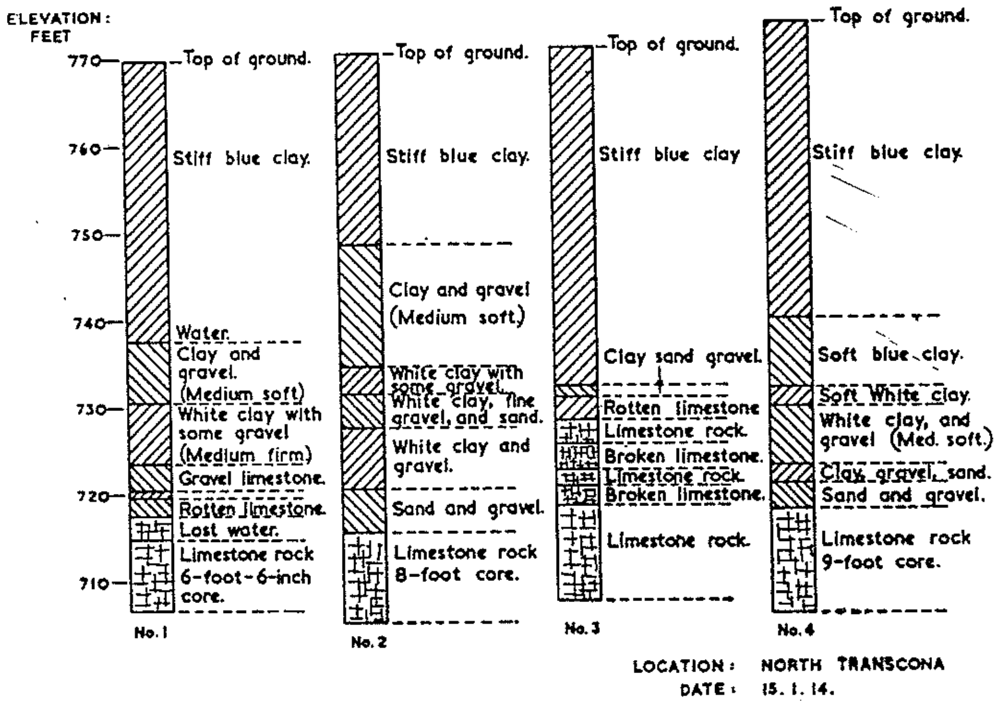
In September 1913, the structure was completed and filling was begun. The grain was distributed as evenly as possible amongst the bins. On October 18, 1913, after 875,000 bushels of wheat had been stored in the elevator, settlement of the bin-house was noted. Within an hour the settlement had increased uniformly to about one foot. This was followed by a tilt toward the west which ceased after 24 hours at an inclination of 26 degrees–53 minutes from the vertical. (Allaire, 1916).

The significance of this failure was realized almost immediately. Early interest centred primarily on the ability of the structure to remain intact during the failure and on the unique underpinning operations. Subsequently, when soil mechanics had provided the basis for computing the ultimate bearing-capacity of various soils, it was realized that the Transcona failure afforded one of the best of the few opportunities for a full-scale check on the validity of such computations.

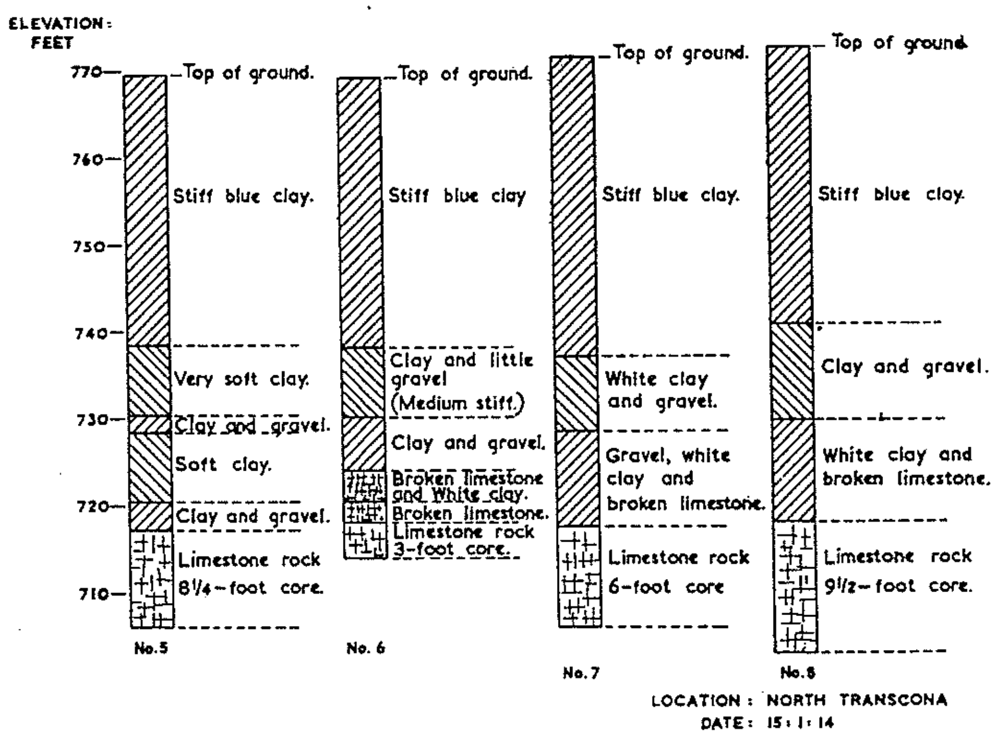
Several wash-borings were made immediately after the failure (Figs 1 (a), and 1 (b)). These indicated that the elevator was underlain by rather uniform deposits of clay. This finding was in agreement with the geological history of the area, according to which extensive fine-grained sediments were deposited in the waters of glacial Lake Agassiz which came into being when the Wisconsin ice-sheet blocked the region's northern outlet. Winnipeg lies above one of the deeper portions of the lake basin and, as a consequence, about 30 to 50 feet of laminated sediments are found overlying the Ordovician limestone bedrock.

Figs 1

(a)



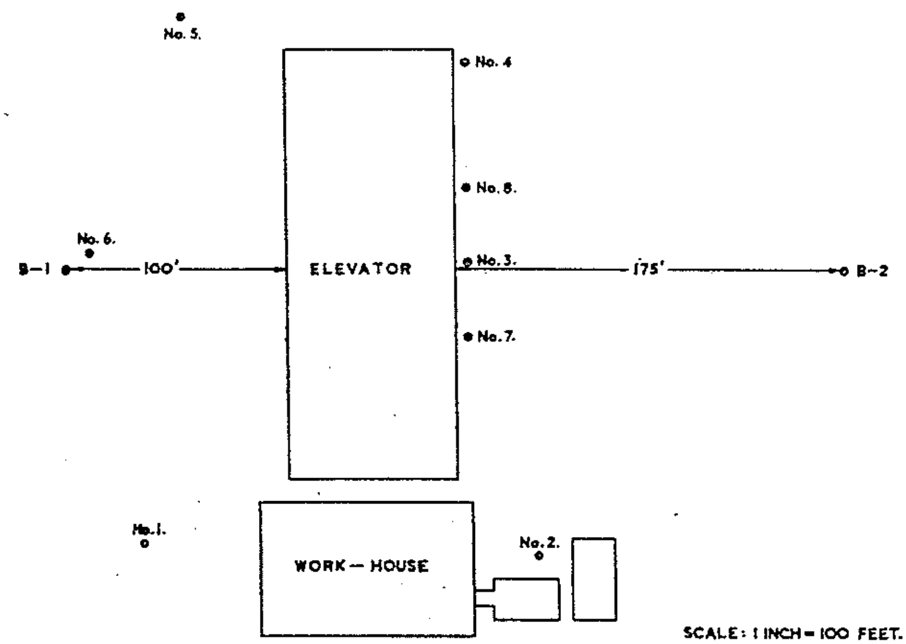
(b)



Canadian Pacific Railway: Records of test holes

In 1951, the opportunity arose to obtain relatively undisturbed samples of the subsoil and to make the tests necessary for a theoretical computation of the bearing capacity. Two borings, B–1 and B–2, were made near to the site at the locations shown in Fig. 2, far enough from the zone of failure to be in material unaffected by the displacements. From each boring, samples in 2-inch thin-walled tubes were taken continuously through the clay strata. Laboratory tests were made to determine the index properties and the stress-strain relationships appropriate for the stability analysis.

Fig. 2



Location of borings

INDEX PROPERTIES

In the laboratory, the samples were cut into specimens 6 inches in length. Each specimen was examined and carefully described, and its natural water-content was determined. Representative samples were selected for determination of the liquid and plastic limits. The limit tests were generally performed on materials that had not been permitted to become air-dried, but a few were performed on oven-dried soil. The oven-drying did not change the results significantly.

The most significant index property, and one that is also directly useful in the stability computation, is the unconfined compressive strength. Each 6-inch specimen was trimmed to a length of 3·5 inches, and tested to failure in unconfined compression by means of a stress-type platform-scale loading machine. The stress on the average cross-section at failure or, if the sample bulged, at 20-per-cent vertical strain was taken as the unconfined compressive strength. The material was then thoroughly remoulded at unaltered water-content and formed into a specimen again 3·5 inches high by $1\frac{7}{8}$ inch diameter, and subjected once more to an unconfined compression test. Typical stress-strain curves for the materials in both undisturbed and remoulded states are shown in Fig. 5.

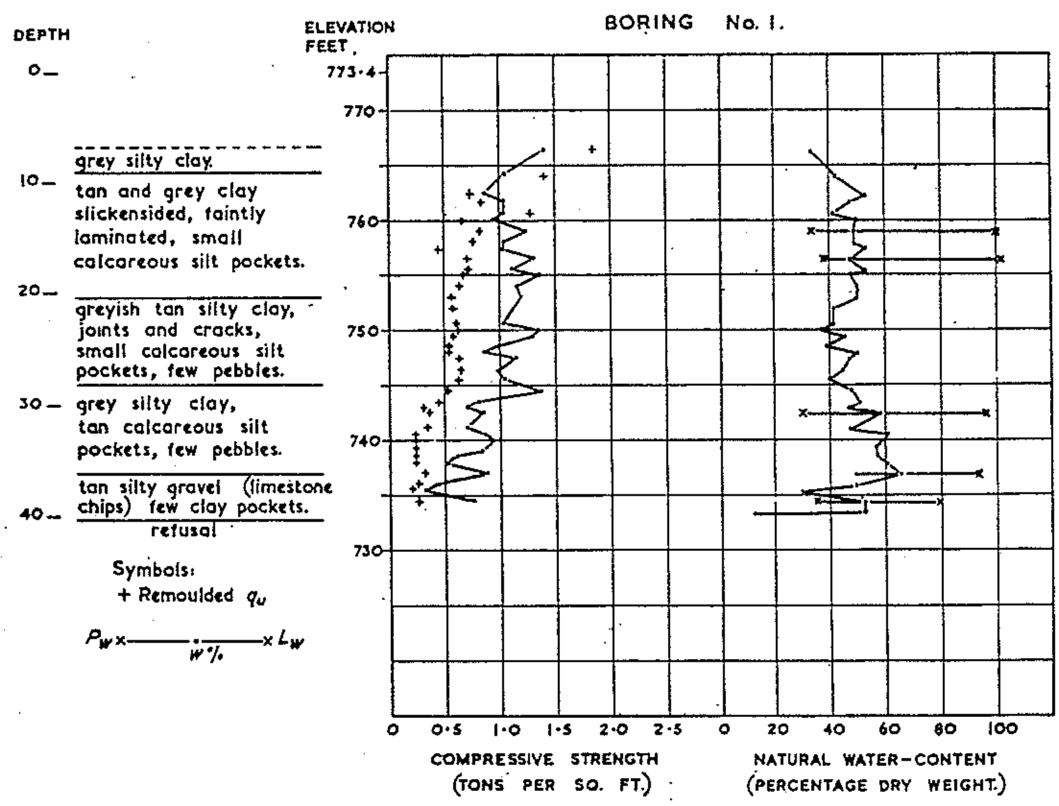
A summary of the descriptions and of the index properties is represented by Figs 3 and 4. From ground level at about El. 772 (feet) to El. 745 the soil is a tan and gray slickensided clay with an average water-content of about 45 per cent, an average unconfined compressive strength of 1·1 ton per square foot, and a sensitivity of about 2. The values of liquid and plastic limit, respectively about 105 and 35 per cent, correspond to an inorganic clay of high plasticity according to the Casagrande plasticity chart. The slickensides have a variable spacing, usually about ¼ inch.

Between El. 745, and El. 737 (B–1), or El. 730 (B–2), is a gray silty clay with an average water-content of about 57 per cent, an unconfined compressive strength of about 0·65 ton per square foot, and a sensitivity of about 2. The Atterberg limits are approximately the same as those of the overlying stratum.

Below the gray silty clay is a tan silty gravel containing limestone chips and pockets of clay. The bedrock is probably represented by refusal at El. 717 (B–2), whereas refusal in B–1 at El. 733 more likely represents a large fragment of limestone. The bedrock was not drilled.

Ground water level was at El. 763 in B–1, 12 hours after the casing was removed.

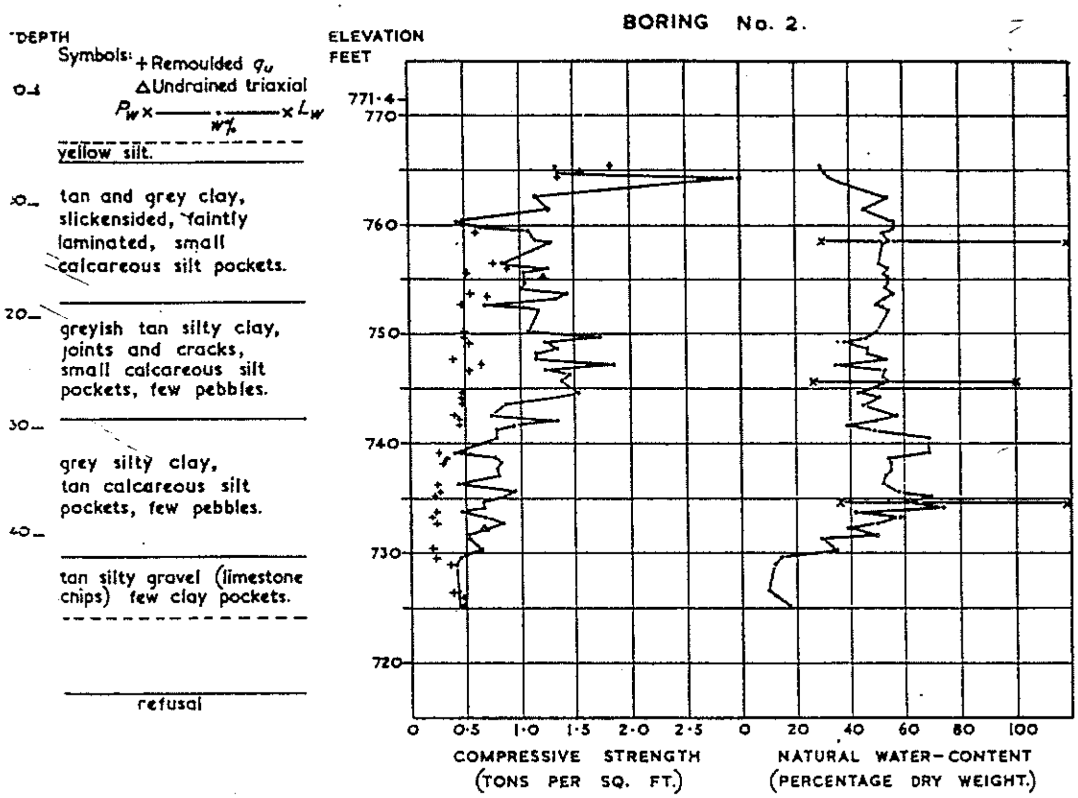
Fig. 3



SUPPLEMENTARY TESTS

Differential thermal analyses on two representative samples were performed to provide an indication of the clay minerals present. About two-thirds of the material were illite, and one-third montmorillonite; very little non-clay material was present. The specific gravity of the solid matter was found to be 2·70.

Fig. 4



A more precise investigation of the stress-strain characteristics in unconfined compression was made by performing tests on representative undisturbed 2-inch samples in constant-strain apparatus. The rate of strain was approximately 0·08 inch per minute. The corresponding stress-strain curves, shown in Fig. 6, indicate that the clays are all fairly elastic.

Two samples were subjected to undrained triaxial tests at a lateral pressure of 2·9 tons per square foot. The difference in principal stresses at failure is shown for these tests by triangular symbols in Fig. 4. The confining pressure appears to have had no influence on the compressive strength, whence it may be inferred that the soil would behave as if $\phi = 0$, and the cohesion or shear strength c would be equal to one-half the compressive strength.

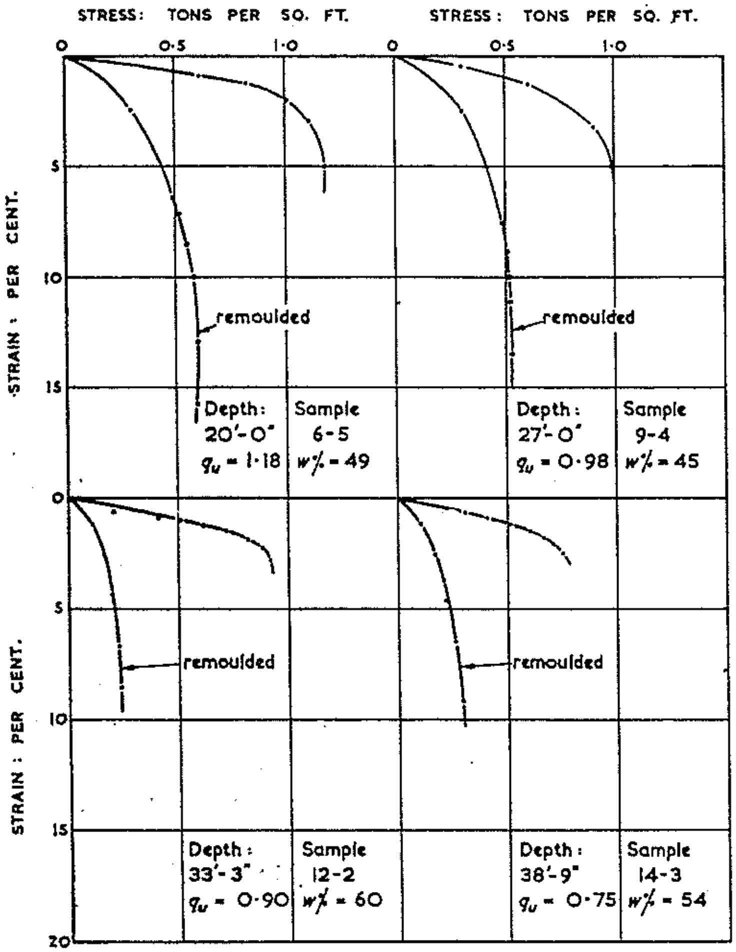
To supplement the description of the soils, a consolidation test was performed on a specimen from the lower part of the tan and gray slickensided clay deposit. The effective over-burden pressure at the level of the specimen was 0·93 ton per square foot. The e-log p curve is shown in Fig. 7 ; it indicates a slight degree of pre-loading, presumably as a result of desiccation.

LOAD AT FAILURE

The load on the base of the elevator at failure is known rather exactly. The elevator contained 875,000 bushels of wheat with a total weight of about 26,000 short tons. The dead load of the structure has been calculated * to be 20,000 tons. The sum of these loads distributed uniformly over the mat (77 feet by 195 feet) represents a unit load of 3·06 tons per square foot. The bottom of the mat was originally established 12 feet below the ground surface. On the assumption that the unit weight of the upper 12 feet of soil was 120 pounds per cubic foot, the reduction in load owing to excavation was 0·72 ton per square foot. Therefore, the net unit-load applied 12 feet below the prairie level was 2·34 tons per square foot.

* Allaire, loc. cit. p. 813.

Fig. 5



CALCULATED BEARING CAPACITY

The net ultimate bearing capacity of a homogeneous clay soil with $\phi = 0$ may be expressed as (Terzaghi, 1943):

$$q_n = cN_c = \tfrac{1}{2}q_u N_c$$

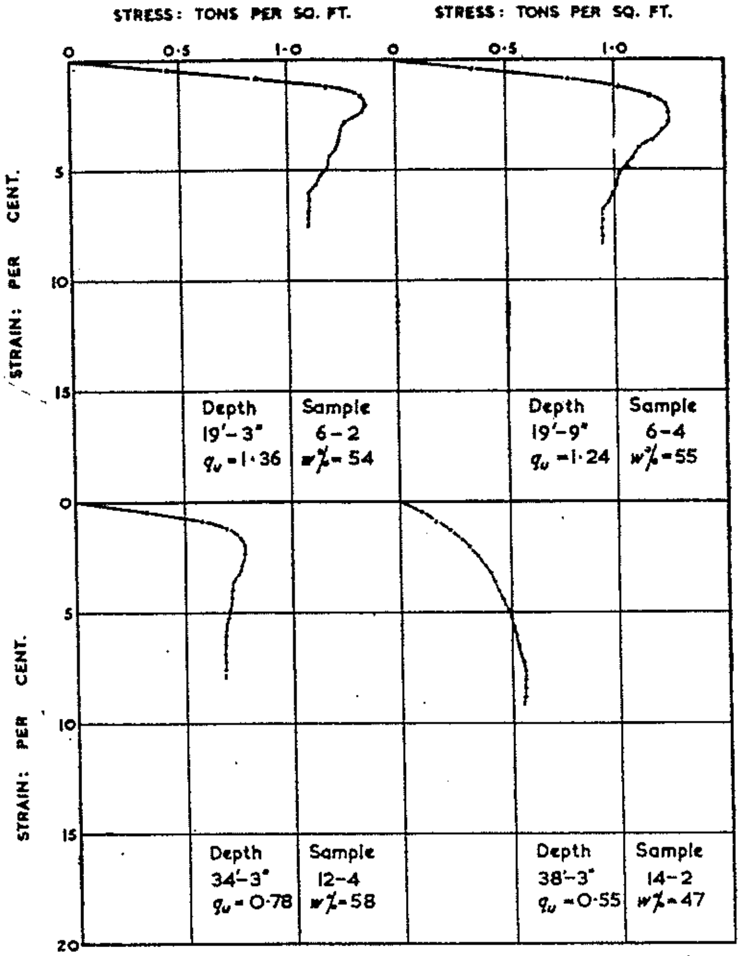
wherein N_c is a factor depending only on the length L and width B of the foundation, and on the depth D of the foundation below ground level. Recent studies (Skempton, 1951) indicate that, where $\frac{D}{B}$ is less than about 2·5, the value of N_c is given with reasonable accuracy as:

$$N_c = 5\left(1 + \frac{B}{5L}\right)\left(1 + \frac{D}{5B}\right).$$

In the case of the Transcona bin-house, $B = 77$ feet, $L = 195$ feet and $D = 12$ feet, whence

$$N_c = 5{\cdot}56.$$

Fig. 6



BORING No. 1.

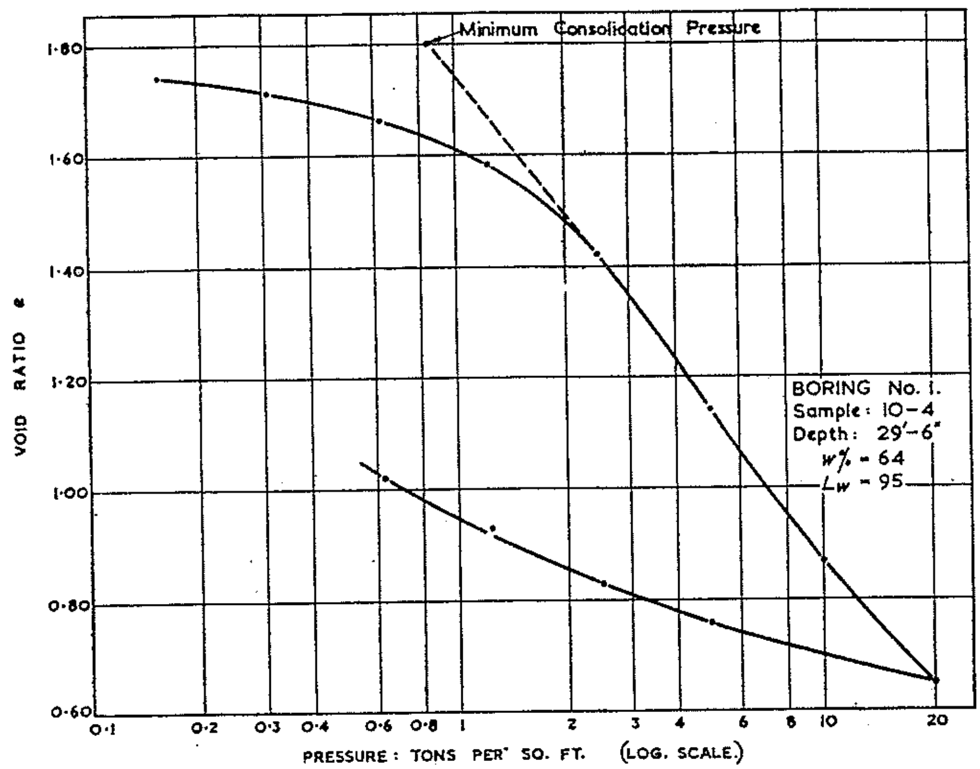
The average value of q_u for the slickensided tan and gray clay between the base of the raft and El. 745 is 1·13 ton per square foot, that of the softer underlying gray clay is 0·65 ton per square foot; and the weighted average for the full thickness of about 35 feet is 0·93 ton per square foot. The bearing capacity of a foundation above a fairly homogeneous clay depends upon the strength of the soil within a depth equal to at least half the width of the foundation, whence it may be inferred that the full thickness of 35 feet would be involved in the failure, and that

$$q_n = 0{\cdot}5 \times 0{\cdot}93 \times 5{\cdot}56 = 2{\cdot}57 \text{ tons per square foot,}$$

a value that compares favourably, for practical purposes, with the actual value of 2·34 tons per square foot. However, this agreement may be somewhat fortuitous because of the uncertainties associated with evaluating the influence of the slickensides in the upper deposit, and the influence of progressive failure owing to the difference in strength and stiffness between the upper and lower clay deposits. Both factors would tend to reduce the bearing capacity below the theoretical value. It is unlikely, however, that the bearing capacity could be as low as that based exclusively upon the strength of the lower layer, or

$$q_n = 0{\cdot}5 \times 0{\cdot}65 \times 5{\cdot}56 = 1{\cdot}80 \text{ ton per square foot.}$$

Fig. 7



A comparison of the preceding values suggests that the influence of progressive failure was actually relatively small, and that the closely spaced slickensides did not introduce a serious error in the estimate of the strength of the upper layer.

ACKNOWLEDGEMENTS

The samples for this investigation were obtained by the Raymond Concrete Pile Company as a part of their research programme. The necessary arrangements in Winnipeg were made by Professor W. F. Riddell, Chairman, Department of Civil Engineering, University of Manitoba. Most of the tests were performed at the University of Illinois, but half the samples from Boring-2 were retained at the University of Manitoba and tested by Messrs A. G. Burrows, and J. K. Maitland, under the direction of Mr Andrew Baracos, Lecturer in Soil Mechanics. The mineralogical analyses were made in the laboratory of Dr Ralph E. Grim, Research Professor of Geology, University of Illinois.

REFERENCES

Allaire, A., 1916. The Failure and Righting of a Million-Bushel Grain Elevator, *Trans. A.S.C.E.*, LXXX 800–803.
Terzaghi, K., 1943. Theoretical Soil Mechanics. *J. Wiley, New York.*
Skempton, A. W., 1951. The Bearing Capacity of Clays, Building Research Congress, I: 180–189.

A. Baracos. The Engineering Journal, Engineering Institute of Canada, Vol. 40, pp. 973-977 and 990.

The Foundation Failure of the Transcona Grain Elevator

A. Baracos, M.E.I.C.

Department of Civil Engineering, University of Manitoba

THE METHODS of soil mechanics development in the past twenty-five years have made possible the determination of the ultimate bearing capacity of soils. The safety factor necessary for sound engineering practice precludes correlation of the ultimate bearing capacity with the analytical determination of the bearing pressures for the impending failure. Thus only on rare occasions when actual failure occurs is a correlation possible. The foundation failure in 1913 of a million-bushel grain elevator at Transcona, a few miles from Winnipeg, Manitoba, provides such an opportunity. It is the purpose of this paper to correlate the data on the foundation failure of the Transcona grain elevator with a recently completed field and laboratory soil mechanics investigation using the latest analytical methods.

At the Building Research Congress held in London, England, in 1951, a notable paper on "The Bearing Capacity of Clays" was presented by Dr. A. W. Skempton[1] The Transcona elevator failure was used by this author as one of his examples and is included in his table of "Field Data on Ultimate Bearing Capacity of Clays". The table, however, was incomplete as presented, with data on actual soil properties missing for two structures, one of them the Transcona elevator.

Another speaker on the same program as Dr. Skempton was R. F. Legget, M.E.I.C., Director of the Division of Building Research of the National Research Council, Ottawa. As he was speaking about "Special Foundation Problems in Canada", Mr. Legget also used the Transcona elevator as an example! In discussion, the desirability of completing the table in Dr. Skempton's paper was stressed. Mr. Legget thereupon promised at the meeting to expedite the study of this foundation failure. This paper prepared cooperatively by the Division of Building Research and the University of Manitoba represents the fulfilment of that promise.

Owing to the necessity of completing other outstanding soil and foundation studies, it was not until late in the summer of 1952 that the work herein described could be started. Only then was it found that some time previously two soil borings had been put down at the site of the elevator under the direction of Professor R. B. Peck, of the University of Illinois. Unfortunately, it was then too late to correlate the proposed Canadian investigation with Dr. Peck's work but it was decided to proceed as planned.

Dr. Peck has now published his results[2], accompanied by an eye-witness account of the failure by Mr. L. Scott White[3]. The investigation now to be described was rather more extensive than the American study, the two being generally complementary, agreement between the various test results being reasonably close, even though carried out quite independently except for a check by the author on a few of the soil samples obtained by Dr. Peck.

Although the foundation failure of the Transcona grain elevator occurred as long ago as 1913, the conditions involved have since frequently been discussed in connection with soil mechanics problems. The author describes a recent investigation made jointly by the Division of Building Research, N.R.C., and the University of Manitoba.

General Description of the Structure

Development of Canada's vast wheat lands in the early part of this century resulted in serious congestion of the Winnipeg railroad yards during peak periods of grain movement. Construction was therefore started in 1911 on the Transcona elevator in conjunction with one of the world's largest railroad yards to facilitate rapid grain movement and to give relief to the Winnipeg railroad yards.

The plan of the elevator is shown in Fig. 1. It consists of a dryer house 18 by 30 by 60 feet high, work-house 70 by 96 by 180 feet high, and the bin house 77 by 195 by 102 feet high, all constructed mainly of reinforced concrete. The foundation failure occurred under the bin house which was designed for storing one million bushels of grain. It consists of 65 circular bins arranged 13 in each of 5 rows running north and south. The 48 interstices between bins are also used for storing grain. A raft foundation, of reinforced concrete 2 feet thick, supports the bins and the conveyor tunnels under the bins. The depth to the bottom of the footings was 12 feet below the ground surface. The design bearing pressure was 6,600 lb. per square foot based on load bearing tests for which the data are no longer available. Dead weight of the bin house was very nearly 20,000 tons.

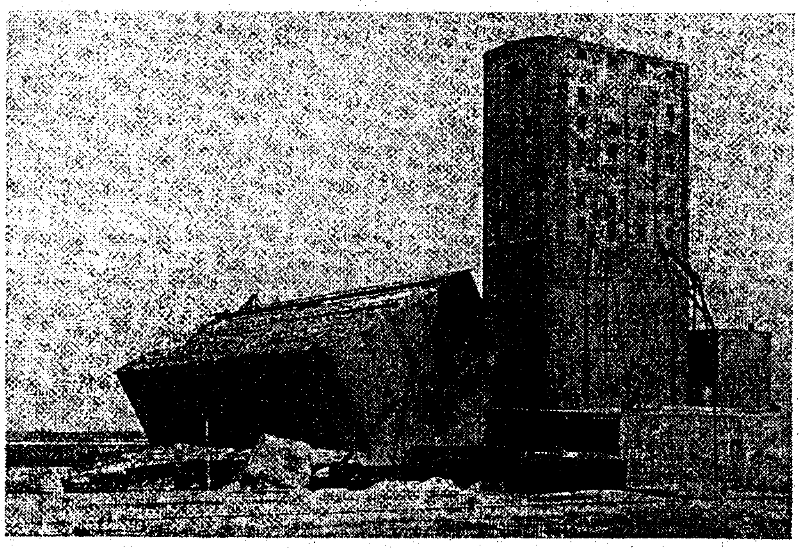
General Description of Soils

Greater Winnipeg lies in the basin of the glacial Lake Agassiz which existed during the recession of the Wisconsin ice sheet. Generally, in this area, the soils may be conveniently grouped as follows. The top 10 feet or less consist of relatively recent deposits of organic soils, flood-deposited silts and silty clays and outwash from higher ground, and modified lacustrine deposits. Under these are found 40 feet or less of glacial lake deposits forming two distinct layers of approximately equal thickness. The top layer is a brown clay and is distinctly varved with many fractional inch-thick layers of silt spaced between layers of clay ¼ inch or more thick. The bottom layer is a grey-coloured clay, softer than the overlying material and having numerous calcareous silt pockets and containing limestone gravel and stones at the greater depths. Beneath the clays are found glacial deposits of rock flour, silts, sands and gravel. The upper portions were deposited as the glacier receded and are underlain by subglacial drift which has been acted on by the full weight of the ice sheet. The subglacial drift is highly consolidated and supports many of the heavier structures in the Winnipeg area. The total thickness of the drift is about 10 feet but varies considerably from this value. The entire area is underlain by Ordovician limestone.

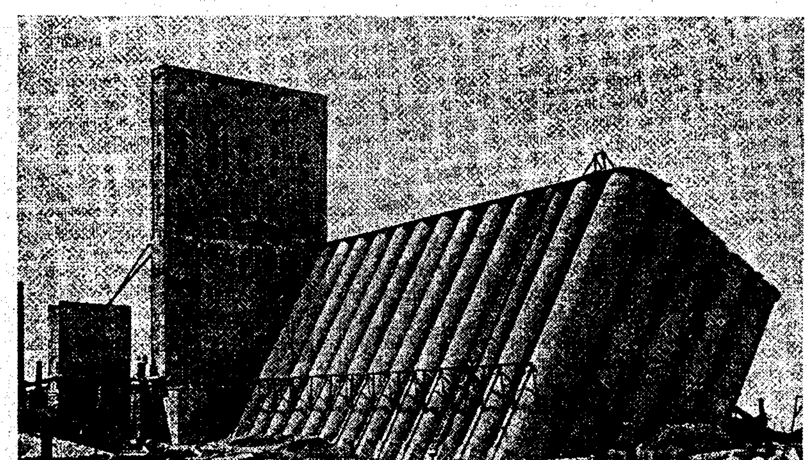
Description of Failure

The storage of grain in the bin house was begun in September, 1913, with considerable care taken to distribute the grain uniformly. On October 18, when 875,000 bushels of wheat were stored, a vertical settlement of a foot was noted within an hour after movement had been detected. The structure then began to tilt to the west and within 24 hours was resting at an angle of 26° 53′ from the vertical and the west side was 24 feet below its original position. The east side had risen 5 feet above original elevation. Eye witness accounts[3,4] stated that the structure acted monolithically with only a few superficial cracks appearing. Its coming to rest, approximately 24 hours after the movement began, corresponded with the cupola falling off the top of the structure.

It was reported that during the failure, the soil around the structure rose to a height of 5 feet above the ground surface around the entire bin house. Photographs taken after the failure show that the greatest upheaval occurred on the west side and was considerably more than 5 feet.



Above: West side of elevator, showing tilt and soil upheaval. Below: East side following foundation failure; early stages of righting operations are shown under way. (Photos: Foundation Company of Canada Limited.)



Calculations based on the dead weight of the bin house, 20,000 tons, and 875,000 bushels of wheat at 60 lb. per bushel, give a unit uniformly distributed pressure of 6,200 lb. per square foot on the clay when failure took place.

The operations to right the structure have been reported in detail by Allaire[5]. The structure has been in successful use since its position was restored.

Field and Laboratory Investigation

Figure 1 shows the location of seven test holes used to obtain samples for the laboratory tests. Holes 4 and 7 were sufficiently removed from the structure to avoid disturbances caused by the failure and the righting operations.

The remaining holes were located nearer the structure, some 60 feet from the bin house, in an effort to ascertain the effects of failure. It was realized, however, that these holes would show the effects of almost 40 years of continued pumping that has taken place since the bin house was righted. Pumping has been necessary to keep the bottom of the bin house dry. After righting, the bin house was approximately 34 feet below the prairie grade. It was not considered practical to place the test holes any closer to the structure as the entire area nearer the building was disturbed by tunnelling, excavation, etc., during the righting operations. To keep the bins dry, a 12-foot-deep trench had, in addition, been excavated around the bin house on all but the south side, further discouraging test holes any closer to the structure than those indicated.

The holes were bored to refusal at a depth between 40 to 50 feet where the dense and coarse glacial deposits were encountered. A diamond drill adapted for taking thin wall Shelby tubes 2 inches in diameter was used for boring and sampling. Samples approximately 2½ feet long were taken at 5-foot intervals or less where changes in soil were evident.

All samples were examined in the laboratory and notes made on colour, stratification, etc. On each sample, moisture contents, density, degree of saturation, and unconfined compression strengths were determined. On representative samples, grain size, Atterberg limits, undrained quick triaxials under constant load increments, specific gravity, and consolidation tests were performed. The unconfined compression and the undrained triaxial tests were performed on undisturbed samples trimmed to 1.5 inches diameter and approximately 3.0 inches long. Both the field and laboratory testing were conducted during the autumn of 1952 and winter of 1952-1953.

Test Results

Typical results of the tests are shown in the Log of test hole 4, Fig. 3. No tests were performed on the material above 10 feet in holes 1, 2, 3, 5, 6, and 7 where fill placed during the righting of the elevator was encountered. Hole 4 showed the silts and silty clays as they probably were over the entire area prior to the excavation for the foundations.

Below the 10-foot level to a depth of 20.5 feet in hole 4, and from 25 to 26 feet in the other holes, a brown highly stratified or varved silty clay was found. The stratification or varves were more or less horizontal and consisted of layers of silt of fractional inch in thickness between closely spaced layers of clay approximately ¼ inch thick. Average test results for this material were as follows:

Unconfined compressive strength (lb./sq.ft.)			2160
Liquid limit			85.3
Plastic limit			29.3
Moisture content (%)			52.4
M.I.T. grain size grouping (%)			
clay	49.4;	silt	42.8;
sand	7.4:	gravel	0.4
Unit weight of soil (lb./cu. ft.)			107

Under the brown silty clay to a depth of 40 to 45 feet from the surface, a highly plastic grey silty clay was found with numerous tan-coloured calcareous silt pockets and limestone pebbles. This material had about the same moisture content as the overlying brown silty clay and a lower unconfined compressive strength. In holes 1, 3, and 7 the bottom few feet of the grey silty clay were found to be very moist and soft. Holes 1 and 7 showed no distinct boundary between the grey silt and the underlying glacial drift. About 3 feet of a mixture of both materials formed a transition layer. Average test results for the grey silty clay excluding the very moist material encountered in holes 1, 3, and 7 and the transition layer were as follows:

Unconfined compressive strength (lb./sq.ft.)			1641
Liquid limit			75.9
Plastic limit			22.8
Moisture content (%)			49.9
M.I.T. grain size grouping (%)			
clay	38.7;	silt	44.5;
sand	13.0;	gravel	3.8
Unit weight of soil (lb./cu. ft.)			110

Because of the wide variation in the bottom grey silty clay and the transition layer, no average values are given.

All the samples below the 10-foot depth showed complete or near complete saturation. Twelve undrained triaxial tests on samples from hole 2 confirmed a negligible angle of internal friction for this type of loading. The consolidation test results (for samples from hole 4) indicate a decrease in compressibility with increased depth. Swelling pressures determined by permitting undisturbed samples to swell under a small load and determining the pressure required to return the sample to its original volume, range from 560 to 2050 lb. per square foot and are typical of the Greater Winnipeg clays which contain about 30 per cent of the more active clay minerals (montmorillonite). Preconsolidation pressures are not accurately determined on these clays but indicate that they are somewhat in excess of overburden pressures probably due to desiccation. The void ratio pressure curves are shown in Fig. 5.

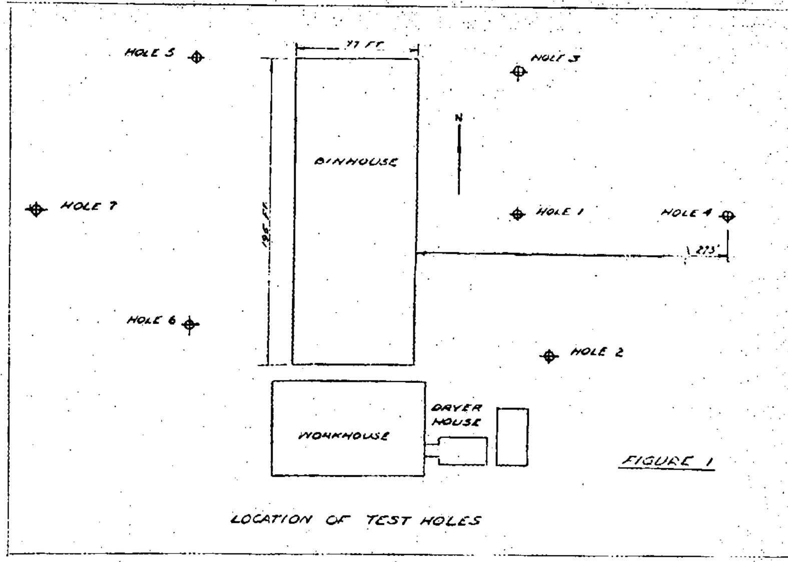
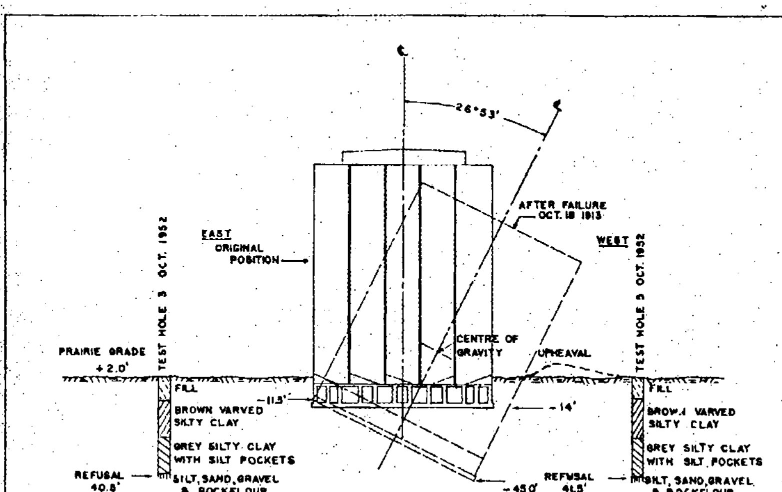


Fig. 1. Plan of the Transcona elevator.



SECTION THROUGH NORTH END OF BINHOUSE BEFORE AND AFTER FAILURE

FIGURE 2

AFTER ALLAIRE, A.S.C.E. TRANSACTIONS FEB.2.1916

The glacial drift was encountered at a depth of 40 to 45 feet. The change from material deposited during the recession of the ice sheet to the subglacial drift appeared to be indicated by a decrease in moisture content approaching or below the plastic limit. Up to 4 feet of the less dense drift were found. Boring refusal was encountered in the subglacial drift corresponding to the depth to which the north end of the bin house settled following the failure. Numerous stones prevented strength and consolidation tests from being performed on the glacial drift. The following data, however, were obtained:

Natural moisture
content range (%) 10.0–13.4
Moist density (lb./cu.ft.) 157. –143
Liquid limit, average 21.0
Plastic limit, average 11.9
M.I.T. grain size grouping (%)
clay (rockflour) 6.0; silt 33.6;
sand 32.3; gravel 28.1

The test holes were not extended to the underlying limestone. Eight test holes bored by the owners of the building have shown, however, that the limestone bedrock was at a depth of approximately 50 feet.

Theoretical Bearing Capacity

The relatively rapid loading of the elevator on saturated clay corresponds to the laboratory undrained quick triaxial test for which the unconfined compression test is a special case. For such conditions it is recognized that the angle of internal friction is negligible and thus the cohesion is equal to half the unconfined compressive strength.

In general the ultimate unit bearing capacity of a soil may be expressed by:

$$q_u = N_c c + N_q y d + N_y y \frac{B}{2} \quad \ldots\ldots (1)$$

where q_u = ultimate unit bearing capacity
c = cohesion
y = unit weight of soil
B = width of footing
d = depth of cover on footing

For long continuous footings, the quantities N_c, N_q, and N_y are pure numbers depending on the angle of internal friction, ϕ. Their values are given in most modern soil mechanics or foundation texts.

For the special case of $\phi = 0$, N_q becomes unity and $N_y = 0$. The equation thus becomes:

$$q_u = N_c c + yd \quad \ldots\ldots\ldots\ldots (2)$$

Prandtl, in an early form of equation (2) evaluated Nc as 5.14 and Terzaghi[6] gives 5.7 for general shear failure and 3.8 arbitrarily for local shear failure. The general shear failure applies when the stress-strain curve (from laboratory tests) is of the type shown in Fig. 4a, or is approached when negligible variation exists in both loading and soil conditions.

For rectangular footings the value of Nc has been shown by analytical methods, model studies and a study of actual failures to be a function of $\frac{L}{B}$ and $\frac{d}{B}$, where L = length of footing. Recently, Skempton[1] has given the following formula:

$$N_c = 5(1 + B/5L) \, . \, (1 + d/5B) \quad \ldots (3)$$

The theory for equation (1) assumes that the soil fails along a composite curve as shown in Fig. 4B. Although the theory is beyond the scope of this report, it may be noted that when $\phi = 0$, the composite curve extends to a depth below the bottom of the footing equal to approximately one-half the footing width. As failure commences, there is a rise of soil on both sides of the footing attributed to "edge action". Complete failure is associated with a further large upheaval on the side to which the building tilts.

Stability Analysis

A general examination of the actual failure and test data shows that the failure was consistent with the bearing capacity theory. The undrained quick triaxial test confirmed a negligible angle of internal friction. The composite curve along which the soil failed would have theoretically extended to a depth equal to about one-half the foundation width or 38½ feet below the bottom of the foundation. Since the dense glacial till occurred at approximately the same depth, it did not prevent the full development of this curve.

It may also be noted that the soil upheaval all around the foundations due to "edge effect" at the start of failure actually occurred. Allaire[5], reports an upheaval of 5 feet. Photographs confirm that further large upheaval consistent with theory occurred on the side to which the structure tilted. The actual direction of tilting is not important as even a very minor eccentricity in loading or variation in soil condition could cause a failure to either side.

The nearest test holes to the structure on the side of tilting were 63 feet distant and from the examination and testing of undisturbed samples, the soil appeared to be unaffected by the failure. Although the failure occurred nearly 40 years ago, it is not believed that the loss in strength of the soil resulting from the failure has been regained. Tests on similar Lake Agassiz deposits[7] do not indicate any extensive thixotropic strength regain for this material. Although no remoulded strength tests were performed, it has been generally found that remoulding results in a loss of one-half of the strength of the Winnipeg clays.

It is also reasonable to assume that because of the nature of the laboratory stress-strain curves and the precautions taken to assure uniform loading of the elevator, that the Terzaghi general shear conditions were satisfied. It is questionable, however, whether the assumption of local shear value (N_c = 3.8) would have been applicable had the stress-strain curves been different.

The undrained quick triaxial test confirmed that the angle of internal friction was negligible and that equa-

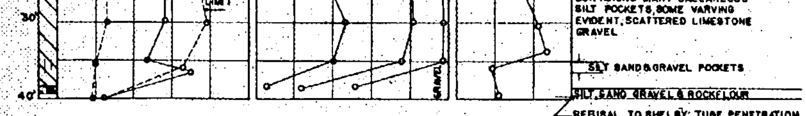


Fig. 3. Typical test hole log. (Test hole 4.)

tion (2) was valid. Substitution in equation (3) with:
$B = 77$, $L = 195$, and $d = 12$ (all in feet) gives $N_c = 5.56$. The ultimate bearing capacity is thus given by:

$$q_u = 5.56c + 12y \quad \ldots\ldots\ldots\ldots \quad (5)$$

It was difficult, however, to ascertain what value of the cohesion should be used in (5). The values for the brown silty clay or the grey silty clay alone would be unjustifiable high and low respectively since the failure plane passed through both materials. Use of the average unconfined compressive strength value of 1850 lb. per square foot for both the brown and grey silty clays from holes 4 and 7 appears the most justifiable. The same value for the remaining test holes 1, 2, 3, 5, and 6, nearer to the building, was 1933 lb. per square foot and probably reflects the effects of consolidation caused by the continuous pumping from under the bin house for a period of almost 40 years. Moisture contents and densities for the grey silty clay when compared for holes 4 and 7 with those of 1, 2, 3, 5, and 6, also indicate the effects of consolidation

	Holes 4, 7	Holes 1, 2, 3, 5, 6
Average moisture content (%)	51.9	48.9
Average moist density (lb./cu.ft.)	107.8	112.2

The average unconfined compressive strength values of 1933 lb. per square foot for holes 1, 2, 3, 5, and 6, and 1850 lb. per square foot for holes 4 and 7, do not include the low values from the 35- to 40-foot depth from holes 1 and 3, and hole 7 respectively. The difference in the values of cohesion, density, and moisture content mentioned, however, are small and could simply reflect statistical accuracy.

Results of substitution in equation (5) are shown in Table I. The unit weight, Y, of the soil covering the footings was taken as 107 lb. per cubic foot and the cohesion as half the unconfined compressive strength.

Discussion

The ultimate theoretical bearing capacity of 6420 lb. per square foot using the most justifiable value of unconfined compressive strength, 1850 lb. per square foot is remarkably close to the actual bearing capacity at failure of 6200 lb. per square foot. The correlation is even better than statistical considerations of the data

(continued on page 990)

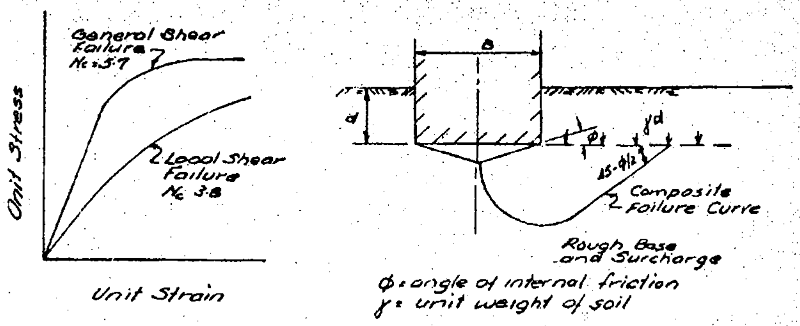


FIGURE 4A
Terzaghi Criteria for N_c

FIGURE 4B
Theoretical Composite Failure Curve

Figure 4.

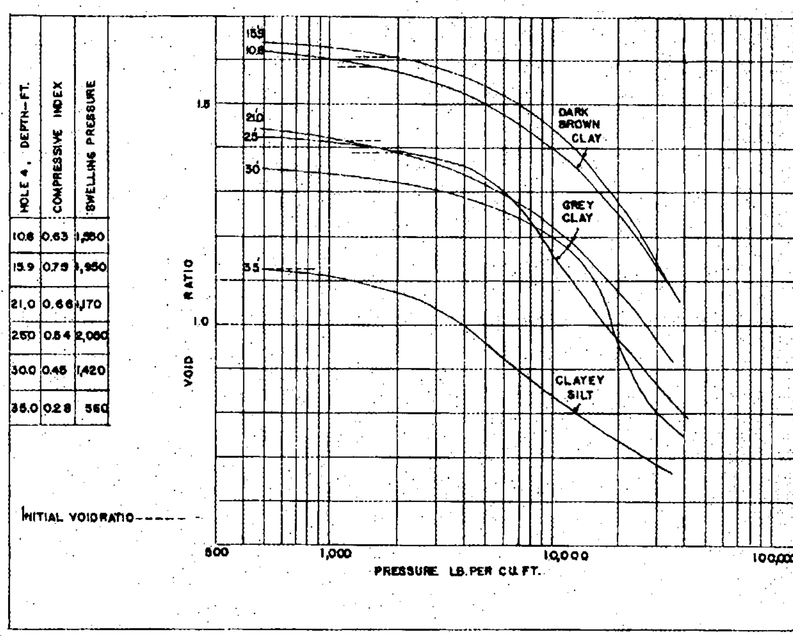


Fig. 5. Consolidation test results, hole 4.

Table I

Based on results for:	Average unconfined compressive strength (lb/sq.ft.)	"q_u" ultimate bearing capacity (lb/sq.ft.)
Brown silty clay — all holes	2160	7280
Grey silty clay — all holes	1641	5840
Brown and grey silty clay		
all holes	1933	6660
holes 1, 2, 3, 5, 6	1960	6730
holes 4, 7	1850	6420

Note: actual ultimate bearing capacity = 6200 lb/sq. ft.

The Transcona Grain Elevator

(continued from page 977)

can substantiate. Reasonable correlation, however, is gained using the other average cohesion values as shown in Table I. This is in spite of such factors as pumping that may have caused soil changes since the failure.

Difficult to explain is the length of time, 24 hours, which elapsed from the time motion began until the building came to rest. The plastic nature of the soils and the gradual transfer of load from the upper stiffer clays to the softer underlying material may be responsible. The slow failure and the varves in the brown clay do not appear to have invalidated the theoretical formula.

To the engineer, it is most reassuring that the study of the Transcona elevator failure and similar studies reported for the foundation failures on clays in widely separated areas, verify the present theories. The advantages of being able to predict the ultimate bearing capacity from a soil study are obvious. With the information now available and the additional studies being made on settlements, foundations on clay may be designed with reasonable knowledge of the safety factors involved and the future behaviour of the structure.

Acknowledgements

The interest in this investigation of the Canadian Pacific Railway, through Mr. R. A. Emerson, chief engineer, and their permission for the sinking of the test holes on their property is gratefully acknowledged. The test borings were put down by the special soil boring crew under Mr. E. Hargest of the Manitoba Department of Public Works. The author is indebted to the Foundation Company of Canada Limited for the photographs which serve as illustrations. The soil testing was carried out by Mr. M. Bozozuk in the soil mechanics laboratory of the University of Manitoba and to him the author records his thanks as also to Dean A. E. Macdonald, M.E.I.C. and to Mr. R. F. Legget, M.E.I.C., Director of the Division of Building Research, National Research Council, with whose approval this paper is published as a joint contribution from the University of Manitoba and the Division.

References

1. Skempton, A. W. The Bearing Capacity of Clays. Building Research Congress, London, I:180-189. 1951.
2. Peck, R. B. and F. G. Bryant. The Bearing Capacity Failure of the Transcona Elevator. **Géotechnique,** III:201-208. 1953.
3. Scott White, L. Transcona Elevator Failure: Eye-witness Account. **Géotechnique,** III:209-214. 1953.
4. Winnipeg Free Press. October 20, 1913.
5. Allaire, A. The Failure and Righting of a Million-bushel Grain Elevator. Trans. Amer. Soc. Civil Engineers. Vol. 80. December 1916.
6. Terzaghi, K. Theoretical Soil Mechanics. Art. 47. John Wiley and Sons, Inc. 1943.
7. Berger, L. and J. Gnaedinger. Thixotropic Strength of Clays. A.S.T.M. Bulletin, September 1949.

Blatz, J. and Skaftfeld, K. 2003. The Transcona grain elevator failure: a modern perspective 90 years later. Proceedings, 56th Canadian Geotechnical Conference, 4th Joint IAH-CNC/CGS Conference, Winnipeg, MB, 29 Sept. - 1 Oct., 2003, pp. 8-22 to 8-29.

THE TRANSCONA GRAIN ELEVATOR FAILURE: A MODERN PERSPECTIVE 90 YEARS LATER

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ABSTRACT
The foundation failure and righting of the Transcona Grain Elevator in 1913 is recognized as a truly remarkable case history made famous by its collapse during loading after bearing pressures exceeded the limiting shear resistance of the underlying clay foundation soil. This paper takes you on a journey lasting 90 years beginning with the construction, failure and righting of the structure. The landmark work carried out in the 1950's comparing the load at failure with that predicted by classical bearing capacity formulae is examined with a modern perspective made possible by finite element modeling techniques using nonlinear effective stress analysis. The results from the effective stress analysis were imported into a limit equilibrium analysis to determine the minimum factor of safety against bearing capacity failure at the failure load with the associated critical slip surface. The vertical settlement and tilting of the structure predicted by the model closely matches the events described by eye witnesses. The time dependency of the porewater pressure generation has been evaluated to explore the possibility that the catastrophic failure could have been avoided using staged loading.

RÉSUMÉ
The foundation failure and righting of the Transcona Grain Elevator in 1913 is recognized as a truly remarkable case history made famous by its collapse during loading after bearing pressures exceeded the limiting shear resistance of the underlying clay foundation soil. This paper takes you on a journey lasting 90 years beginning with the construction, failure and righting of the structure. The landmark work carried out in the 1950's comparing the load at failure with that predicted by classical bearing capacity formulae is examined with a modern perspective made possible by finite element modeling techniques using nonlinear effective stress analysis. The results from the effective stress analysis were imported into a limit equilibrium analysis to determine the minimum factor of safety against bearing capacity failure at the failure load with the associated critical slip surface. The vertical settlement and tilting of the structure predicted by the model closely matches the events described by eye witnesses. The time dependency of the porewater pressure generation has been evaluated to explore the possibility that the catastrophic failure could have been avoided using staged loading.

1. INTRODUCTION

Bearing capacity theory is relatively well understood by today's geotechnical engineers. While many foundations on cohesive soils are still designed using classical total stress bearing capacity theory first proposed by Terzaghi (1943), designers now have more sophisticated analysis tools at their disposal allowing them to carry out advanced effective stress analysis. However, at the turn of the 20th century no such formulations or tools existed and by necessity, local experience was relied upon to design foundations. Such was the case of the Transcona Grain Elevator in Winnipeg, a structure made famous by its collapse during loading after bearing pressures exceeded the limiting shear resistance of the underlying clay foundation soil. Since settlement is often the controlling factor in design, cases of ultimate shear failure are uncommon today, in particular for large structures. While it is almost certain that the mat foundation for the Transcona Grain Elevator was designed to tolerate large settlements, its susceptibility to a deep-seated base shear failure was neither understood nor expected.

The significance of the failure did not escape early Foundation Engineers who recognized this unique opportunity to compare the loading at failure with that predicted by classical bearing capacity formulae. Results presented by Skempton (1951) included the Transcona Grain Elevator as one of the examples outlining a comparison of calculated bearing capacity factors with cases where failure was observed and therefore the factor of safety was known to be unity. The first geotechnical evaluation of the Transcona Grain Elevator failure was reported in Peck and Byrant (1953) where a limited site investigation was undertaken. R.F. Legget presented the results as Director of Building Research of the National Research Council during a presentation on "Special Foundation Problems in Canada" and during subsequent discussions, promised to study the foundation failure in more detail (Baracos 1955). In fulfillment of that promise, a joint study including detailed test holes and laboratory testing was undertaken by Baracos (1957). The conclusions from this study compared well with results presented in Peck and Byrant (1953) where the ultimate theoretical bearing capacity of 6,420 psf was remarkably close to the actual observed bearing capacity at failure of 6,200 psf.

Fast forward to the 21st century and the opportunity to evaluate this failure using effective stress analysis that for the first time examines the time dependency of porewater pressure generation and explores the possibility that the catastrophic failure could have been avoided using staged loading. This paper takes you on a journey lasting 90 years beginning with the construction, failure and righting of the structure, a story that in itself illustrates the ingenuity of Patrick Burke-Gaffney, an engineer trained in Ireland, whose first assignment in Canada was that of Instrumentman, in charge of raising the Transcona Grain Elevator. The historical perspective based on the landmark work carried out in the 1950's is described and compared with the modern perspective made possible using finite element analysis techniques. The results of modern day analysis and the general lessons learned from this paper however, cannot overshadow those taught to us through the resourcefulness and determination of the men who righted the structure using nothing more than fish scales to model the loading along rows of piers and picks and shovels to excavate soil from beneath the tilted structure.

2. HISTORIC PERSPECTIVE

The Transcona Grain Elevator consists of a sixteen story workhouse and a ten story binhouse connected by two conveyer tunnels as illustrated in Figure 1. On October 18 1913, grain transfer into the newly constructed 1,000,000 bushel Transcona Grain Elevator was well underway when the binhouse began to settle and tilt to the west.

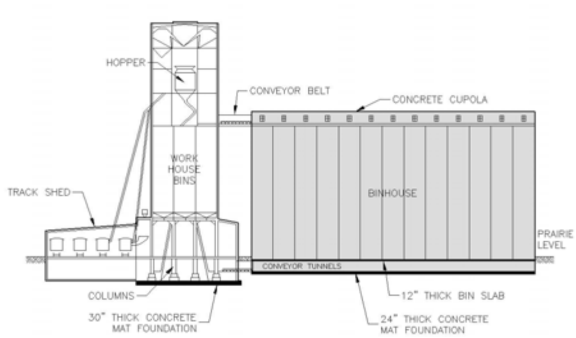

Figure 1. Profile of the workhouse and binhouse.

Within 24 hours, the structure came to rest at an angle of 27 degrees from vertical as shown in Figure 2. In its final position, the west edge of the mat foundation was 24 feet below its original elevation and the east edge had risen about 5 feet. Earth mounds as high as 15 feet surrounded the structure, having been thrust up as the settlement occurred. Almost unbelievably, the monolithic concrete structure, with the exception of the concrete cupola was intact and the first order of business was to tap each of the 65 bins to salvage their valuable contents. Fortunately, the workhouse on the south side of the bins was only slightly cracked as the bins settled and did not itself experience any subsidence. The replacement cost of the binhouse was estimated between $140,000 and $150,000 and according to Mr. J.G. Sullivan, then Chief District Engineer for CP Rail, "it was doubtful if any effort would be made to restore the elevator".

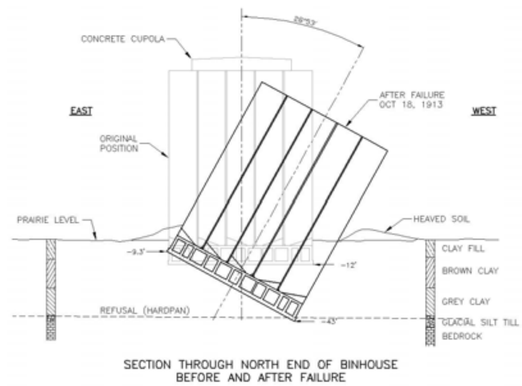

Figure 2. Cross-section of failed binhouse.

Figure 3 shows the final configuration of the structure following failure.

Figure 3. Binhouse after failure.

In December 1913, the Foundation Company of Montreal and Vancouver submitted a plan to Canadian Pacific Rail to underpin the workhouse as it was feared its foundation might also fail. The plan was accepted and work began almost immediately to underpin the structure by sinking a pier under each building column. Because of the heavy loads and height of the structure, it was first necessary to install an elaborate system of internal and external timber shoring. Despite significant groundwater intrusion into the 5 foot diameter piers (Chicago Wells) that were all excavated by hand, the workhouse operations were completed by the beginning of June 1914.

During the workhouse underpinning, the Foundation Company convinced CP Rail that it was possible to salvage the binhouse by righting it and underpinning the structure once the vertical position had been reached. The structure was to be righted by excavating under the high (east) side and gradually lowering the mat foundation to the elevation of the low (west) side. Initially, a trench was excavated along the entire east side of the binhouse to the underside of the mat foundation as shown in Figure 4.

Figure 4 Excavation of the east side of the binhouse.

Drifts were then excavated beneath the mat foundation and a row of 14 piers was sunk to bedrock along the west edge of the mat. The intent was to support the structure with these piers acting as a fulcrum, about which the structure would be rotated as the soil was removed from under the high side. As construction proceeded, the original plan was modified as shown in Figure 5.

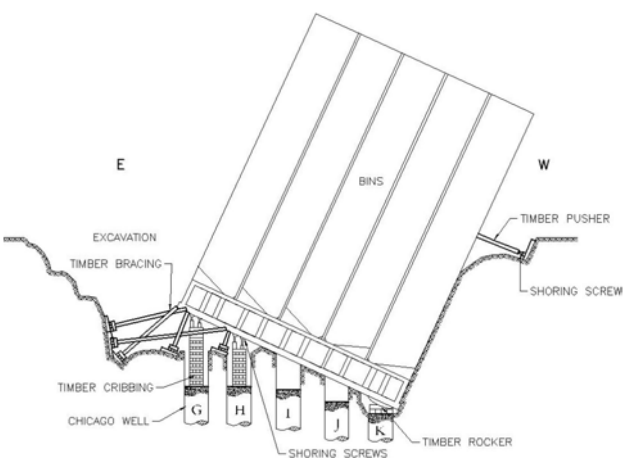

Figure 5. Original plan for righting binhouse.

The structure was raised on the west side using shoring screws and timber rockers installed on the tops of successive rows of piers (Figure 6). To assist in the righting, twelve timber pushers were placed against the west side of the bins. On October 17, 1914, two days behind schedule, the binhouse was back in its vertical position having been raised about 12 feet in the process. Figure 7 shows the structure after the completion of the righting and underpinning operations. The structure has been successfully used since this time and is now owned and operated by Parrish and Heimbecker Limited.

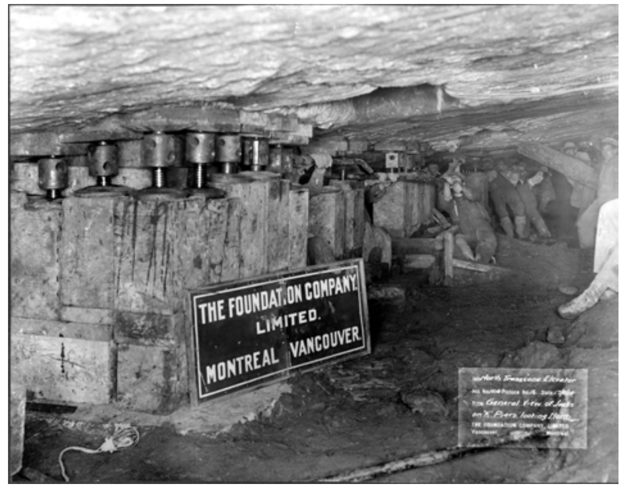

Figure 6. Shoring screws used to lift binhouse.

Figure 7. Binhouse after righting.

3. SOIL PROFILE AND INDEX PROPERTIES

Two soil borings were put down at the site in 1951 (Peck and Byrant 1953). In 1952, six additional borings were made by Baracos (1955). The soil profile around the elevator interpreted from these borehole logs consists of clay fill to a depth of about 10 feet, stiff brown clay to a depth of about 25 feet and highly plastic grey clay to a depth of 40 to 45 feet from ground surface. The clay deposits are lacustrine material deposited by glacial Lake Agassiz in the immediate post-glacial period when the Wisconsin Ice sheet blocked the region’s northern outlet. The clay is underlain by 10 to 15 feet of glacial silt till that becomes increasingly dense with depth (hardpan).

Table 1. Soil Index Properties

Soil Property	Lacustrine Clay		Glacial Silt Till
	Upper Brown	Lower Grey	
% Clay	49.4	38.7	6.0 (rockflour)
% Silt	42.8	44.5	33.6
% Sand	7.4	13.0	32.3
% Gravel	0.4	3.8	28.1
Field Moisture Content (%)	52.4	49.9	Range: 10 – 13.4
Liquid Limit (%)	85.3	75.9	21.0
Plastic Limit (%)	29.0	22.8	11.9
Bulk Unit Weight (pcf)	107	110	Range: 143 – 157
Unconfined Compressive Strength (psf)	2160	1641	Not Measured

Limestone bedrock is encountered at a depth of approximately 45 to 50 feet. The upper limestone is heavily fractured and water bearing. The Index properties of the clay foundation soils, averaged from laboratory test results carried out on samples from the 1952 investigation are summarized in Table 1. The clay was saturated below a depth of 10 feet. Compression indices range from 0.45 to 0.75 as reported by Baracos (1955). Overconsolidation ratios generally decrease with increasing depth, ranging from as high as 8 in the brown clay to 1.5 in the underlying grey clay. These observations of overconsolidation are consistent with local experience and are believed to be primarily a result of desiccation. Within the grey clay, it is almost certain that the preconsolidation pressures were exceeded by the foundation load, a stress level at which large consolidation settlements of the structure would be expected.

4. MODERN PERSPECTIVE

Finite element software for routine geotechnical engineering analysis has been available since the early 1990's. With the rapid development of the personal computer, the use of software in engineering practice has quickly grown to its current state. Since these tools are relatively new, there are many foundation failure case histories that have never been analyzed using these modern day tools. The foundation failure of the Transcona Grain Elevator is an excellent example of a case history widely recognized as having being used to validate the total stress bearing capacity solution. This paper details a first attempt (to the authors' knowledge) to model the foundation failure using a fully coupled finite element effective stress consolidation analysis. The failure is then analyzed using stress and porewater pressure conditions from the effective stress analysis imported into a limit equilibrium application to examine the factor of safety at the failure load conditions. The model is not rigorous in its development but is intended to allow global exploration of the failure mechanism and the effective stress conditions that existed during loading and failure.

4.1 Loading Conditions

Knowing the dead weight of the structure (20,000 tons) and the weight of the grain in the bins permits the foundation pressure at the time of failure to be determined. Based on a unit weight of grain of 60 pounds per bushel, the foundation pressure when excessive settlement was first observed is estimated to be 6,200 psf. Although it was reported that the bins were uniformly filled, the possibility of eccentric loading cannot be overlooked. Even a small eccentric load (say in the order of 3 feet differential grain level in the bins) could have significantly affected contact pressures beneath the mat in particular if the structure is considered to be rigid (Nordlund and Deere 1970). For simplification, previous total stress analyses were carried out assuming uniform contact pressures. The analysis presented in this paper the mat was modeled using structural elements to incorporate the rigidity of the mat and as a result the non-uniform contact pressure distributions.

4.2 Model Definition

The first step in the modeling process was to gather historic information on the soil and groundwater properties at the site. Excellent data was available from reports and papers presented by Baracos (1976), Mishtak (1964), and Allaire (1916). The geometry and stratigraphy for the Transcona Elevator is relatively simple in form. The foundation geometry is well defined and the soil borings undertaken at the site show that the depth to the till is relatively consistent over the area affected by the loading. The model was developed using Seep/W[1] coupled with Sigma/W[1] to analyze the time-dependent porewater pressure response due to the appied total stress at the surface. The factor of safety at selected times was then analyzed using the limit equilibrium package Slope/W[1].

The original grain elevator foundation level was 5.2 m below ground surface leaving 10.1 m of clay between the foundation and the underlying glacial till.

[1] Geoslope International, Calgary, Alberta, Canada

Table 2: Constitutive Model Parameters

Layer (Depth)	Layer1 (0 – 1.8 m)	Layer 2 (1.8 – 4.1 m)	Layer 3 (4.1 – 6.9 m)	Layer 4 (6.9 – 10 m)
Constitutive Model	Linear Elastic	Modified Cam-clay	Modified Cam-clay	Modified Cam-clay
E (kPa)	20,000	-	-	-
ν	0.4	0.4	0.4	0.4
OCR	-	7	3	1.1
λ	-	0.5	0.4	0.3
κ	-	0.01	0.01	0.01
Γ	-	3.75	3.75	3.75
M	-	0.58	0.58	0.58

To simplify the finite element model, the model surface was set to be coincident with the original design elevation for the base of the mat foundation. The initial condition (prior to construction) was defined using constant head boundary conditions of 2 m below prairie level. The hydraulic conductivity of the clay was defined using typical values for Winnipeg clay and results provided by Baracos (1976). Although Baracos (1976) showed slight anisotropy in the hydraulic conductivity values (horizontal and vertical), for simplicity an isotropic hydraulic conductivity value of 1×10^{-8} m/s was assumed. The same hydraulic boundary conditions were maintained for the loading phase of the model.

For the stress deformation component of the model, stress-strain properties were taken from laboratory measurements provided by Allaire (1916) and Baracos (1955) in addition to local experience. The east and west boundaries were set far enough from the edge of the footing to ensure that they would not influence the stress or strain fields resulting from the applied loading. Figure 8 shows the Sigma/W domain with the four soil units, applied vertical stress loading and the structural elements used to define the properties of the rigid concrete mat. The east and west boundaries were set as zero displacement in the horizontal direction and the base horizontal boundary was specified as zero vertical displacement. Unit weights were applied to establish the in-situ stresses and a surcharge load was applied to represent the overburden soil above the foundation level that was excluded from the finite element mesh. This simplificaton ignores the shear strength of the soil above the foundation elevation. Although the strength of the brown clay within this upper horizon is significantly greater than the soil underlying the foundation, it can be argued that because of desiccation, only very small shear resistance could have been developed in this layer and therefore, its omission would have little affect on the results.

As expected, the veritical and horizontal stress distirbutions are hydrostatic as a function of the unit weight of the soil materials. For the transient model, the boundary conditions remained the same and the load was applied as a constant increase from day zero to time 30 days at the maximum load of 300 kPa. The footing was represented using structural elements with very high values of moment of inertia and stiffness to ensure rigid response under applied loading. The upper 1.8 m was defined as a linear elastic material to avoid numerical instabilty at the corners of the rigid footing (excessive shear and tension stresses) followed by three layers of modified Cam-clay material. Table 2 outlines the model stratigraphy including the final soil properties used in the model. Once the time dependent consolidation model had been run, the stress and porewater pressure distribution was imported into Slope/W to examine the factor of safety against bearing capacity at a specific time. The strength parameters were the same as those used in the modified Cam-clay model and the minimum failure surface was searched for using a grid of radius points.

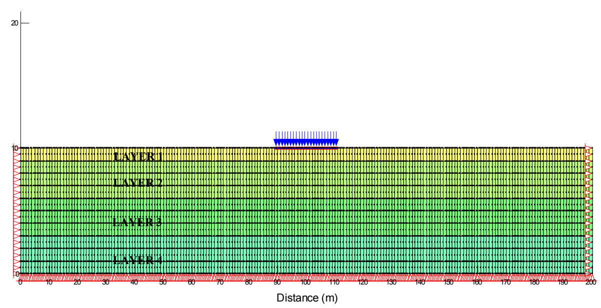

Figure 8. Finite element model domain.

5. MODEL RESULTS

The model was first calibrated using the behaviour of the failed structure. Soil properties were modified during the initial runs to calibrate both the factor of safety at the failure load (300 kPa) and the vertical displacements at the point of incipient collapse. Figure 9 shows the porewater pressure profile versus depth under the center of the mat at increasing time intervals. Constant values of porewater pressure at the surface and underlying till are representative of the boundary conditions applied in the Seep/W model. As the total stress increases with time, so does the porewater pressures under the mat, reaching a maximum value at approximately 8 m corresponding to the softest clay unit (OCR of 1.1) just above the clay till interface. Figure 10 shows the porewater pressure distribution below the mat at failure corresponding to a surface load of 300 kPa. The 300 kPa contour represents the zone of maximum porewater pressure (308 kPa) with values decreasing to approximately hydrostatic at the edge of the zone of influence from the loading.

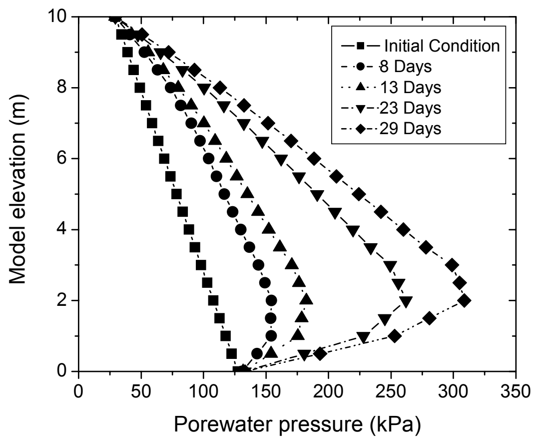

Figure 9. Porewater pressure distribution below mat.

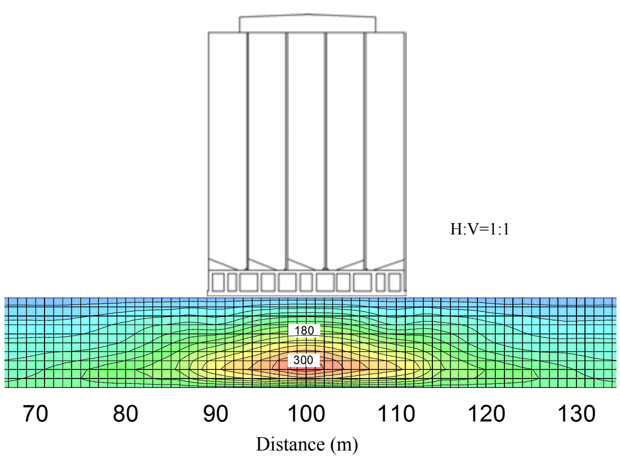

Figure 10. Porewater pressure distribution at failure.

Figure 11 shows the vertical surface settlement profile for different time steps. It is interesting to note that the vertical setttlements are relatively uniform until approximately 18 days after the initiation of loading at which time rotation of the footing initiates. The maximum unform settlements in the model match well with the 0.3 m observed settlements (White 1953) providing confidence that the model is representing the observed behaviour. The transition from uniform vertical displacements to a rotational displacement pattern suggests a progression from consolidation settlements to a bearing capacity failure. The model became unstable after the 29th day time step. Although this is not a clear indicator of instability (due to catastrophic shear failure) it can be interpreted as the development of an unstable loading condition.

Figure 12 shows the failure surface associated with the minimum factor of safety (FS) corresponding to the stress and porewater pressure conditions at the maximum load conditions (300 kPa). The factor of safety contours show that good convergance to the minimum factor of safety has been achieved. The foundation was noted to settle vertically at the west side and rise on the east side confirming that rotation about a point inside the edge of the footing occurred. The failure surface in the model was also noted to extend to the softer clay at the clay till interface. The observation that the majority of the sliding surface occurs under the foundation is consistent with porewater pressures increasing beneath the foundation. This also corresponds to the region where the lowest effective stresses would exist.

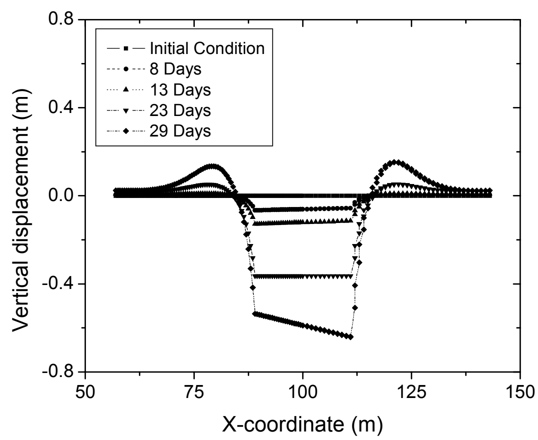

Figure 11. Surface profile of mat during loading and at incipient failure.

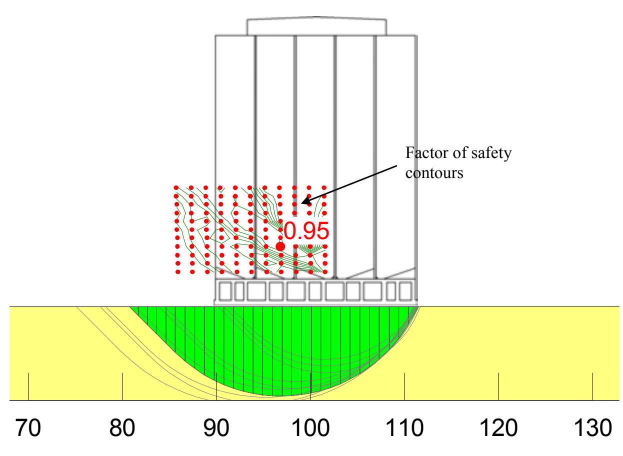

Figure 12. Critical failure surface.

6. LESSONS LEARNED

It came as a surprise to the engineers at the time that the Transcona Elevator failed considering that it had been designed with bearing pressures consistent with those used for similarly loaded shallow foundations for major structures in the City of Winnipeg. Plate bearing tests carried out at the base of the excavation (12 x 12 inch plate) demonstrated that bearing pressures as high as 8,000 to 10,000 psf could be safely applied (Morley 1996). Similar results were achieved from tests conducted during construction of the Shoal Lake Aqueduct in 1916 (City of Winnipeg Historic Drawing A356). These results confirmed engineers' beliefs that satisfactory performance could be expected at bearing pressures that are now recognized to be well in excess of those required to ensure serviceability.

The major difference between the Transcona Grain Elevator and many other shallow foundations is the foundation breadth. Since the breadth of the Transcona Grain Elevator mat is very large in comparison to conventional spread footings (and the plate loading tests), the depth of influence for the Transcona Grain Elevator was much larger. Penetration of the zone of influence to the softer clay above the glacial till interface provided a preferential zone of weakness for shear failure and also a more compressible zone for vertical settlements. Conceptually, this observation might support the development of progressive failure in which the maximum shear stresses are not mobilized simultaneously. Figure 13 shows the vertical strain distribution below the foundation at the failure load. The maximum strains (and therefore compression) occur in the soft clay directly above the till interface. This is also the zone where the maximum porewater pressure increase occurred due to the compressibility of the soil.

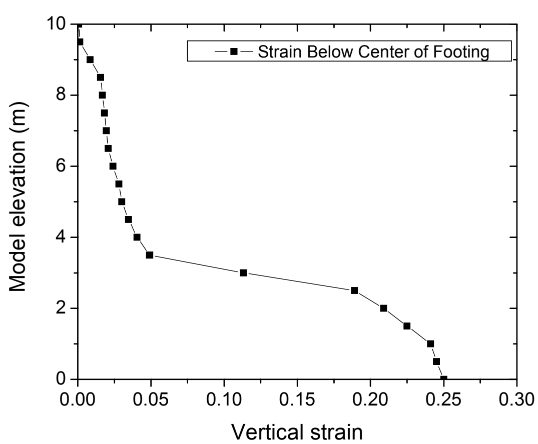

Figure 13. Strain profile below center of mat at maximum pressure.

Beyond simply understanding why the failure occurred, the fully coupled consolidation model provides the opportunity to ask the question, 'could the elevator have been filled successfully by staging the loading to ensure that the stress paths below the foundation did not reach the shear failure condition?'. To answer this question, staged loading was modeled by determining the time of loading where the stress and porewater pressure conditions corresponded to a factor of safety of 1.2 in the limit equilibrium model. The loading at that point was then held constant for a period of one month to allow for porewater pressure dissipation (and corresponding consolidation) and then loading was recommenced to the final design value (380 kPa). Figure 14 shows the porewater pressure at depths of 4.1 m and 8.2 m below the center of the mat for the original calibration model (to failure) and the staged model designed to examine an alternative loading function. As shown in the figure, porewater pressures in the original model (solid symbols) increase at both depths with increasing load up to a time of 28 days when failure occurred.

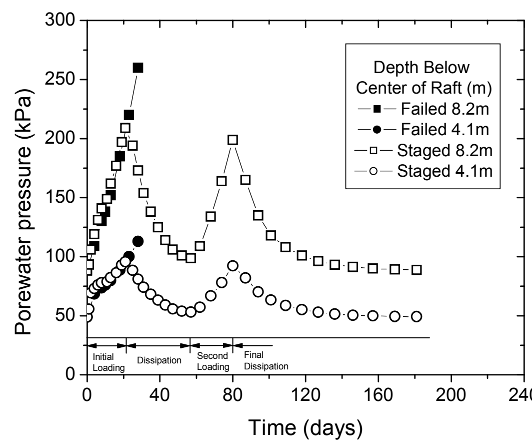

Figure 14. Porewater pressure at two points under the center of the mat.

The initial porewater pressure response matches the staged model up until approximately 20 days when the loading was stopped to allow for porewater pressure dissipation. Following the end of loading, the staged model shows decreasing porewater pressure due to dissipation until approximately three months when the loading is again initiated. The porewater pressures then increase until the maximum stress is reached (380 kPa) however, the porewater pressures never exceed those reached in the first stage of loading indicating the factor of safety was greater than 1.2 for all time steps in the second loading stage. The final time of six months shows nearly complete dissipation of excess porewater pressures corresponding to a stable condition.

7. CONCLUSIONS

Foundation failures can be summarized as an unacceptable difference between expected and observed performance (Morley 1996). In the case of the Transcona Grain Elevator, the observed performance was seen not only as an unacceptable event but also as an unexplainable event. While today's standard of practice would have easily predicted the outcome, such was not the case in the early 1900's when an observational approach was in many cases, the best available analytical tool. It was a time when a lack of understanding of soil behaviour could be offset by reacting to problems with ingenuity and determination. It was in many respects a unique classroom that provided an opportunity to observe such failures rather than trying to visualize them. The engineers of the day truly believed that every precaution had been taken to prevent such an event from occurring. Prior to any investigations on the property, J.G. Sullivan, Chief Engineer for CP Rail reported that it was believed that the 'earth was solid' and therefore, the presence of unsuspected soft soil was the reason for the failure (Winnipeg Free Press 1913). He reiterated the fact that the foundation soil had been tested at twice the weight under which the elevator collapsed. By the 1950's the mechanics of the failure were understood and the lessons learned from the event provided validation of classical bearing capacity formulae. It is also now understood that the interpretation of the results from small scale plate bearing tests mislead the designers as it does not mimic the zone of stress influence that the mat foundation imposed on the clay underlying the horizon where the tests were carried out.

This paper has attempted to take the forensic investigation of the failure to a new level of understanding. While the modeling did not reveal any unexpected results, it provides an example of the ability to analyze foundation performance using the integration of a number of commonly used modern day tools. The coupled groundwater flow and deformation model allowed the failure to be analyzed using non-linear effective stresses. It reliably modeled the porewater pressure generation and dissipation in the foundation soil in response to the external load from filling the grain bins. The results reflect the engineering properties of the soil, in particular the presence of the soft clay underlying a heavily overconsolidated upper clay horizon. The maximum porewater pressure increases and vertical strains occurred in this layer due to its compressible nature. The vertical settlement profile predicted by the model closely matches the events described by eye witnesses (White 1953). The transition from uniform settlements to a rotational displacement suggests a progression from consolidation to a bearing capacity failure.

If one assumes that serviceability of the elevator could have been maintained even with unavoidable vertical settlements from the loading, the model demonstrates that staged loading could have been employed to reduce the likelihood of catastrophic bearing capacity failure which occurred. Using a design factor of safety of 1.2, the bins could have been safely loaded to about 60 percent of their capacity over a one month period before allowing approximately one month for dissipation of excess pore water pressures. The bins could then have been loaded to their maximum capacity over the third month and if left loaded, excess pore water pressures would have been completely dissipated three months hence. Given the rigidity of the foundation, this postulation relies on perfectly concentric loading and uniform soil conditions, conditions which arguably may not have been possible or may not exist. It does however suggest that it may have been possible to avoid what at the time was explained as an act of God. Although more than a foot of settlement may have occurred at that point, the ingenuity displayed in righting the structure leaves no doubt CP Rail would have modified the operation of the elevator accordingly.

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REFERENCES

Allaire, A. (1916) The failure and righting of a million – bushel grain elevator. Transactions of the ASCE, 80, 799-832.

Baracos, A. (1955). The foundation failure of the Transcona Grain Elevator. Report No. 45 of the Division of Building Research, National Research Council of Canada, Ottawa, Canada.

Baracos, A. (1957). The foundation failure of the Transcona Grain Elevator. Engrg. J., Montreal, Quebec, Canada, 40(7), 973-977.

Baracos, A. (1976). Compositional and structural anisotropy of Winnipeg soils a study based on scanning electron microscopy and X-ray diffraction analyses. Canadian Geotechnical Journal, 14, pp. 125-137.

Mishtak, J. (1964). Soil mechanics aspects of the Red River Floodway. Canadian Geotechnical Journal, I(3), 133-146.

Morley, J. (1996). "Acts of God": The symbolic and technical significance of foundation failures. Journal of Performance of Constructed Facilities, Vol. 10, No. 1, pp. 23-31.

Norlund, R.L. and Deere, D.U. (1970) Collapse of Fargo Grain Elevator. Journal of the Soil Mechanics and Foundations Division, ASCE, SM 2, pp. 585-606.

Peck, R.B. and Byrant, F.G. (1953). The bearing-capacity failure of the Transcona Elevator. Geotechnique, London, England, 3(1), 201-208.

Skempton, A.W. (1951). The bearing capacity of clays. Building Research Congress papers Presented in Division I. Building Research Congress. London, England, pp.180-189.

Terzaghi, K., (1943). Theoretical Soil Mechanics. J. Wiley and Sons inc., New York. Pp. 386-392.

White, L.S. (1953). Transcona Elevator failure: eye-witness account. Geotechnique, London, England, Vol. 3, pp. 209-214.

Winnipeg Free Press. October 20, 1913.

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References

Allaire, A. (1916) The failure and righting of a million –bushel grain elevator. Transactions of the ASCE, 80, 799-832.

Baracos, A. (1955). The foundation failure of the Transcona Grain Elevator. Report No. 45 of the Division of Building Research, National Research Council of Canada, Ottawa, Canada.

Baracos, A. (1957). The foundation failure of the Transcona Grain Elevator. The Engineering Journal, Engineering Institute of Canada, Vol. 40, 973-977 and 990.

Baracos, A. (1976). Compositional and structural anisotropy of Winnipeg soils a study based on scanning electron microscopy and X-ray diffraction analyses. Canadian Geotechnical Journal, 14, pp. 125-137.

Blatz, J. and Skaftfeld, K. (2003). The Transcona Grain Elevator failure: A modern perspective 90 years later, Proceedings, 56th Canadian Geotechnical Conference, Winnipeg MB, September 2003, 8-22 to 8-29.

Manitoba Free Press. October 20, 1913.

Mishtak, J. (1964). Soil mechanics aspects of the Red River Floodway. Canadian Geotechnical Journal, I(3), 133-146.

Morley, J. (1996). "Acts of God": The symbolic and technical significance of foundation failures. Journal of Performance of Constructed Facilities, Vol. 10, No. 1, pp. 23-31.

Norlund, R.L. and Deere, D.U. (1970) Collapse of Fargo Grain Elevator. Journal of the Soil Mechanics and Foundations Division, ASCE, SM 2, pp. 585-606.

Peck, R.B. and Byrant, F.G. (1953). The bearing-capacity failure of the Transcona Elevator. Geotechnique, London, England, 3(1), 201-208.

Skempton, A.W. (1951). The bearing capacity of clays. Building Research Congress papers Presented in Division I. Building Research Congress. London, England, pp.180-189.

Terzaghi, K., (1943). Theoretical Soil Mechanics. J. Wiley and Sons inc., New York. Pp. 386-392.

White, L.S. (1953). Transcona Elevator failure: eye-witness account. Geotechnique, London, England, Vol. 3, pp. 209-214.