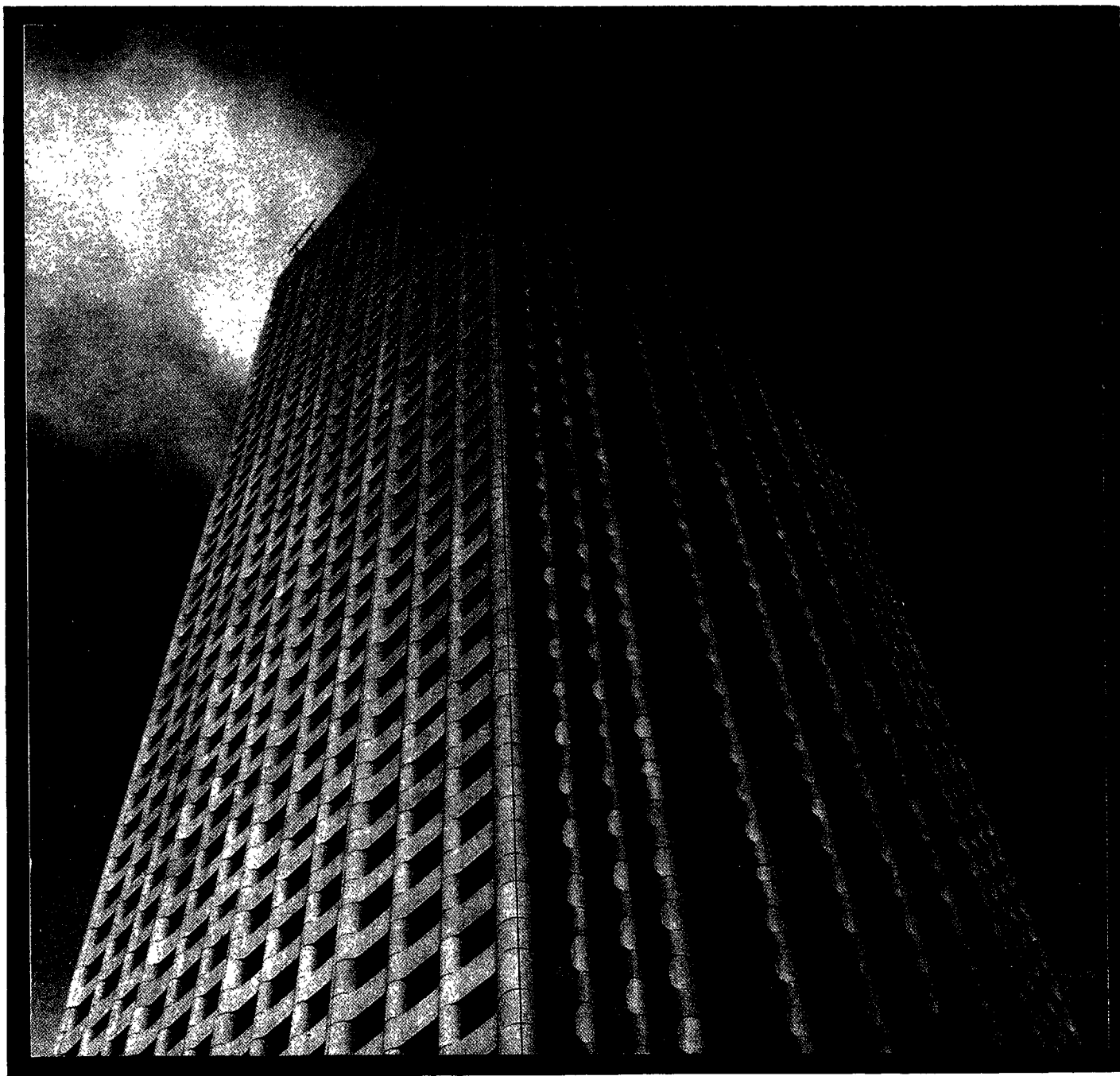


SEISMIC DESIGN AND CONSTRUCTION



Compilation 31

American Concrete Institute

Seismic Design and Construction

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Preface

This ACI Compilation combines material previously published in CONCRETE INTERNATIONAL Magazine to provide compact and ready information on a specific topic. The material in such a compilation does not necessarily represent the opinion of any ACI technical committee — only the opinions of the authors of the compiled articles. However, the material presented here is considered to contain useful information for readers interested in the subject.

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Chairman, ACI Committee 341

Earthquake-Resistant Concrete Bridges

John W. Wallace

Chairman, ACI Committee 368

Earthquake Resisting Concrete Structural Elements and Systems

Todd Perbix

Chairman, ACI Committee 369

Seismic Repair and Rehabilitation



On the cover: A relatively new addition to the skyline of Minneapolis-St. Paul is the City Center/Multi-Foods Tower. This 52-story building required 400,000 sq ft of sand-blasted precast panels. Architect: Skidmore Owings & Merrill; general contractor: PCL; and precast contractor: Gage Brothers.

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Rehabilitation of the Jordan River Concrete Buttress Dam for Seismic Loads

by Tibor J. Pataky and Bradley G. Kemp

The Jordan River Dam (Fig. 1) is a part of the Jordan River Power Development on Vancouver Island, approximately 50 km west of Victoria, B.C., Canada.

Three types of dam were considered during the planning stage in 1912: concrete gravity, masonry gravity, and reinforced concrete buttress. Because of the shortage of naturally available and readily usable construction materials a decision was made to build an Ambursen type concrete buttress dam. The dam was constructed between August 1912 and October 1913.^{1,2}

Under B.C. Hydro's current dam safety program, comprehensive inspections and reviews (CIRs) are carried out once every six years for high hazard dams. Such a CIR was carried out for the Jordan River Dam in 1985 and the results reported in Reference 3. The major recommendations of the CIR report were to:

- Complete the foundation investigations started in 1985
- Assess condition and strength of the

concrete in the buttresses and the upstream slab

- Complete the dynamic analyses started in 1985

Evaluation of existing structure

Field non-destructive testing

Ultrasonic pulse velocity (UPV) testing of Buttresses 38 and 40 was carried out in July 1986. UPV readings were taken at a grid of about 1.2 m. A total of 192 readings were taken for Buttress 38 and 136 for Buttress 40. A histogram of UPV readings is shown on Fig. 2. A rather subjective interpretation of quality would be that a good portion of the concrete is of acceptable quality, but 20 to 25 percent of the UPV readings imply poor quality concrete.

The UPV readings were correlated with compressive strength by conducting tests on samples prepared from 15 cores. The relationship between UPV on dry specimens and compressive strength is shown on Fig. 3. The general tendency for the UPV to be in direct

proportion to compressive strength is quite apparent, but the scatter is wide, as indicated by a correlation coefficient of 0.32. The degree of saturation had a negligible effect on the readings.

Laboratory tests on concrete

Laboratory tests were conducted to obtain information on the physical properties of concrete such as compressive, shear, and tensile strengths, modulus of elasticity, Poisson's ratio, thermal coefficient of expansion, water absorption, and air void content.

Compressive strength tests were carried out in accordance with ASTM C42-84a. Thirty-one 146 mm diameter horizontal cores were drilled; of these, 28 were taken perpendicular to and 3 in the plane of the buttress. Strengths varied from a low of 6.25 MPa to a high of 38.7 MPa, with a mean of 18.1 MPa.

To determine the shear strength of the concrete at horizontal construction joints, which appeared to be a potential source of weakness, twenty 203 mm diameter cores were taken with their axis

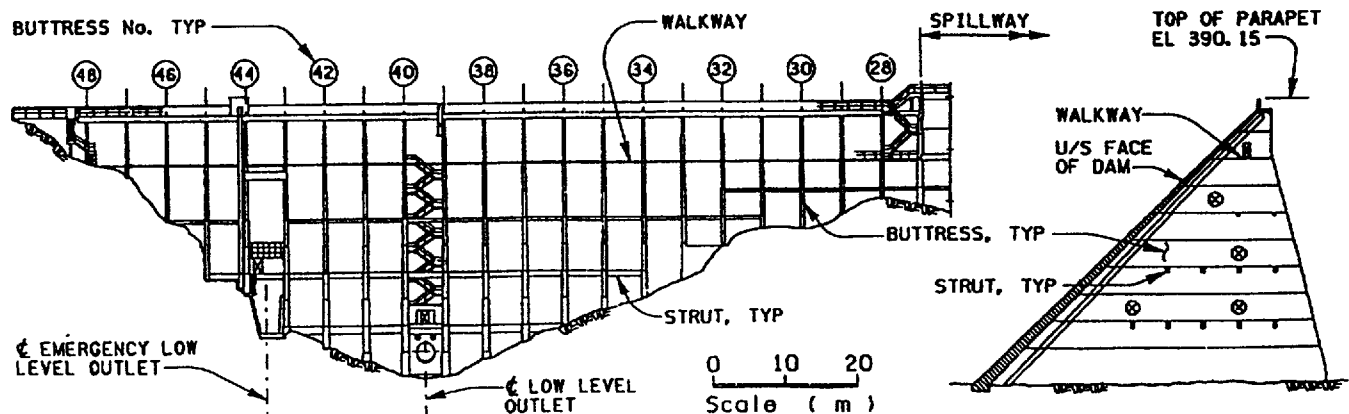


Fig. 1 — Jordan River Dam prior to rehabilitation.

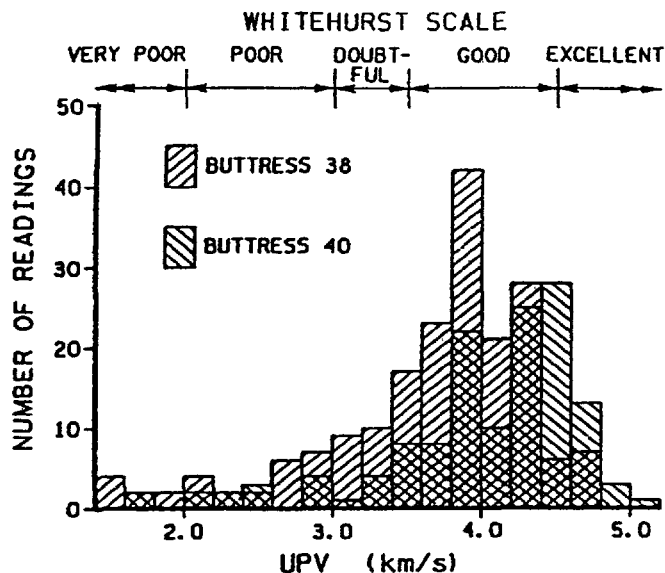


Fig. 2 — Histogram of ultrasonic pulse velocity readings.

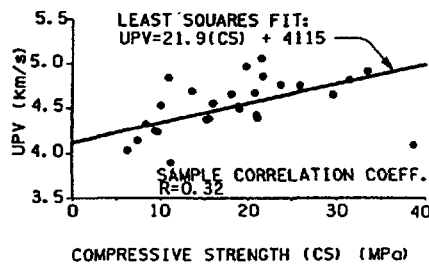


Fig. 3 — Relationship between ultrasonic pulse velocity and compressive strength.

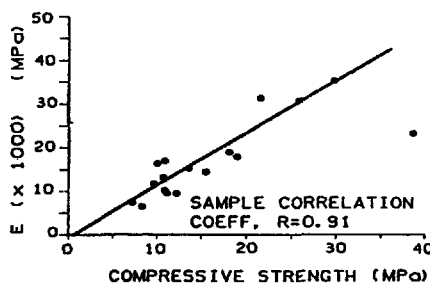


Fig. 5 — Relationship between modulus of elasticity and compressive strength.

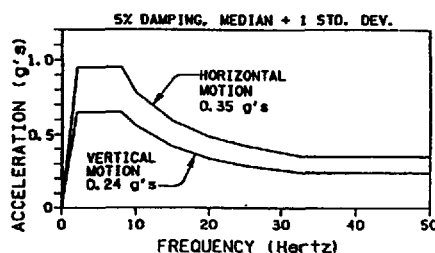


Fig. 6 — Response spectrum.

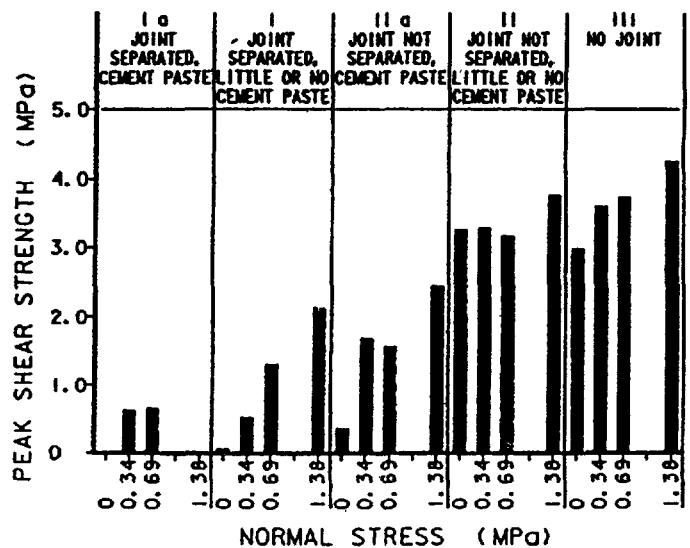


Fig. 4 — Peak shear strengths for various joint classifications.

horizontal but perpendicular to the planes of the buttresses. From these cores 36 samples were prepared for direct shear testing. For joint classification and the recorded peak strengths see Fig. 4. It was observed that an overabundance of cement paste along a joint produced two major effects:

- Overall strength was reduced
- Cohesion component of strength was eliminated

Concrete samples without joints showed a remarkably high shear strength of 3.0 to 4.2 MPa. Those with joints (and cement paste) did not, and all of their shear strength seems to have been derived from friction.

The tensile strength of ten samples, based on the ASTM C496-85 tests, averaged 1.75 MPa which is about 10 percent of the mean compressive strength mentioned above.

The modulus of elasticity tests were carried out in accordance with ASTM C469-83 on 30 samples. Both dial and strain gauges were used in the measurement of strain. Good linear relationship was found between compressive strength and modulus, with a correlation coefficient of 0.91 (Fig. 5).

Other relevant buttress concrete properties were:

- Average poisons ratio = 0.19
- Average thermal coefficient of expansion = 9.15×10^{-6} /degree C
- Dry bulk specific gravity = 2.37
- Average air void content = 0.9 percent

Tests on the concrete for the upstream slab of the buttress dam indicated much

better quality than in the buttresses. Compressive strengths were between 18.8 and 32.6 MPa, with an average of 25.5 MPa.

Dynamic analyses of the existing buttress dam

The vulnerability of buttress dams to cross-canyon earthquake-generated loads was recognized during the California Dam Safety program in the late 1960s: "Nearly all of these dams were constructed with inadequate bracing."⁵ Others have recognized this inherent weakness in buttress dams.^{6,7}

Initial analyses were carried out using simple pseudo-static analyses with increasing prototype reality built into the structural models as exemplified in Reference 8. Pseudo-static analyses were soon required to be replaced by dynamic analyses regardless of the results.⁹ A good example of dynamic analysis can be found in Reference 11.

The existing structure was analyzed both for upstream-downstream and cross-canyon earthquake-induced ground motion. Peak ground acceleration values were 0.18 g and 0.35 g for the design basis earthquake (DBE) and maximum credible earthquake (MCE) respectively. Concurrent peak vertical ground motion values were assumed to be two-thirds of the horizontal peaks.

The Newmark-Hall Response Spectrum,¹² scaled to 0.35 g with 3 percent and 5 percent damping ratios, was used to define structural response.

Cross-canyon analyses with the assumption that the existing struts were capable of developing full tension and

compressive dynamic loads indicated the struts would fail. The tallest unsupported buttresses would deflect laterally 320 mm and would have bending stresses of about 14 MPa. Not even modern concrete could withstand such tensile stresses without cracking and since the buttresses are virtually unreinforced, collapse of the dam would be inevitable.

Upstream-downstream analyses, which included hydrodynamic effects, indicated moderate tensile stresses along the upstream and downstream edges of the buttress. It was postulated that although these stresses could result in cracking collapse of the dam would not likely occur. The results of these analyses are reported in Reference 10.

Rehabilitation alternatives

Failure of the dam, with the reservoir at the normal maximum elevation, would result in breaching of a power intake dam, flooding of Jordan River Generating Station, and rapid and extensive flooding of a log sorting area and the community of Jordan River. Loss of a number of lives could be expected under these conditions.

To reduce the potential hazard of the existing dam, a number of alternative remedial measures have been considered, including:

- Strengthening the existing buttresses and struts (\$8.5 x 10⁶)
- Infilling the spaces not occupied by the buttresses with concrete to El. 378.0 (\$10.3 x 10⁶)
- Infilling work rock fill to El. 378.0 (\$9.6 x 10⁶)
- Infilling with concrete and extending its top to El. 390.1 (\$12.0 x 10⁶)
- Building a new concrete gravity dam (\$14.8 x 10⁶)

There were non-structural solutions considered as well, such as abandoning the dam or relocating downstream residents, however these were considered neither socially nor economically acceptable.

In September 1988 a decision was made to strengthen the existing buttresses with a series of ductile reinforced concrete frames consisting of pilasters and continuous struts. The struts would be capable of resisting and delivering to the foundation all earthquake-induced cyclic tensile and compressive forces.

Design of rehabilitation

Initial layout of new strengthening frames was based on pseudo-static calculations. Final response spectrum analysis and frame optimization utilized the ANSYS-PC/LINEAR 4.3 finite element software package on an IBM-compatible PC. Several sensitivity studies were completed to test analysis assumptions.

Design loads

Mass densities for concrete and water were assumed to be 2,400 kg/m³ and 1,000 kg/m³ respectively. Reservoir hydrostatic pressures were based on El. 386.2, the crest of the free overflow section. Thermal effects were based on a mean annual temperature variation of 15 C.

Dynamic analyses used 0.35 g and 0.24 g peak ground accelerations for horizontal and vertical motions respectively.³ Amplifications of ground motion was based on the Newmark-Hall response spectrum¹² for 5 percent damping (Fig. 6).

Hydrodynamic effects in the upstream-downstream direction were allowed for by attaching to the upstream edge of the finite element model lumped masses derived from Zanger's hydrodynamic pressure¹³ on a sloped surface. Hydrodynamic effects were not considered in the cross-canyon direction.

Computer models

Single buttress model

A single buttress model was used for both the cross-valley and upstream-downstream seismic motions. Refinement of the strut/pilaster layout and design of the new upstream-downstream strengthening members were also carried out with the single buttress model.

The model of the tallest buttress consisted of 1076 eight-noded isoparametric solid elements with 3 degrees of freedom at each node. A total of 500 master degrees of freedom were used.

Two-dimensional frame model

Two dimensional frame models were used for cross-valley dynamic analysis and thermal loading. All new cross-valley strengthening members were designed using the output from these models.

The upstream slab of the dam and the buttress haunch were modelled using 1 dimensional mass elements. A total of 2584 plate elements were used in the model with 500 master degrees of freedom.

Sensitivity studies

Spring stiffness

The input value of spring stiffnesses to represent the elastic support provided by the struts on the single buttress model was found to have a significant impact on vibration frequencies and stresses. Several unit load methods were studied for obtaining spring stiffnesses, including placement of unit loads at:

- One strut/buttress intersection
- Coincident at all strut/buttress intersections along a given strut line
- Coincident at all strut/buttress intersections

The latter method was eventually considered as best to represent strut stiffness during dynamic loading.

Mesh density

Knowledge of seismic-generated tensile stresses in the unsupported buttress concrete between frames was used to optimize the frame layout. A study was completed using models with variable mesh densities in the buttress concrete between pilasters. A relationship between maximum tensile stress between pilasters and mesh coarseness was then developed, as it was found that as the mesh was made finer, a limiting value of the maximum tensile stress between pilasters was attained. Model-generated stresses were then factored to account for mesh density.

Element type

To conserve on computing time, plate elements were investigated for use in the single buttress model. For a comparative study, two models were constructed, one of plate elements and one of isoparametric elements. Support conditions were kept similar for both models. The results of cross-valley analyses showed the plate element model produced modal vibration frequencies which were on the average 1.5 Hz lower. Since plate elements were also found to not effectively model thickened areas of the structure, such as the buttress haunch and the new pilasters, use of them was not considered further.

Damping ratio

Initial analyses were completed with a 3 percent damping ratio for the critical cross-valley excitation. It was later decided to use a 5 percent damping ratio to better reflect the inherent energy dissipating ability of concrete undergoing cracking. This resulted in a 15 to 20 percent reduction in dynamic stresses and reactions.

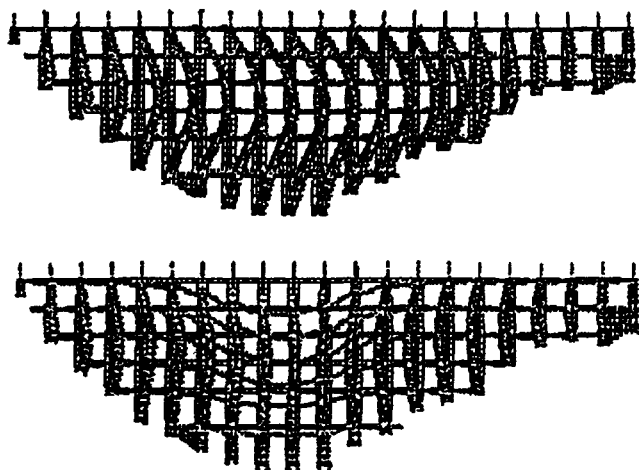


Fig. 7 — Two-dimensional frame model, deflected shape for first significant mode. above: Cross-valley excitation; below: Vertical excitation.

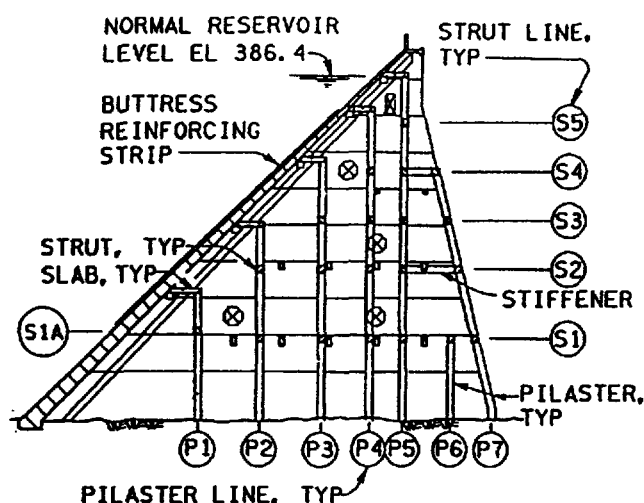


Fig. 8 — General arrangement of new strengthening elements on tallest buttress.

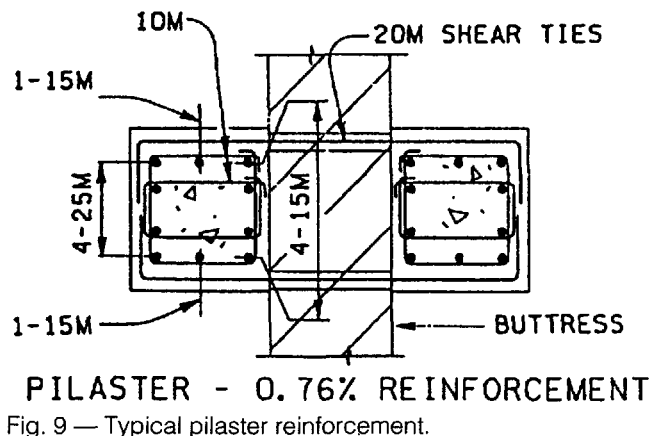


Fig. 9 — Typical pilaster reinforcement.

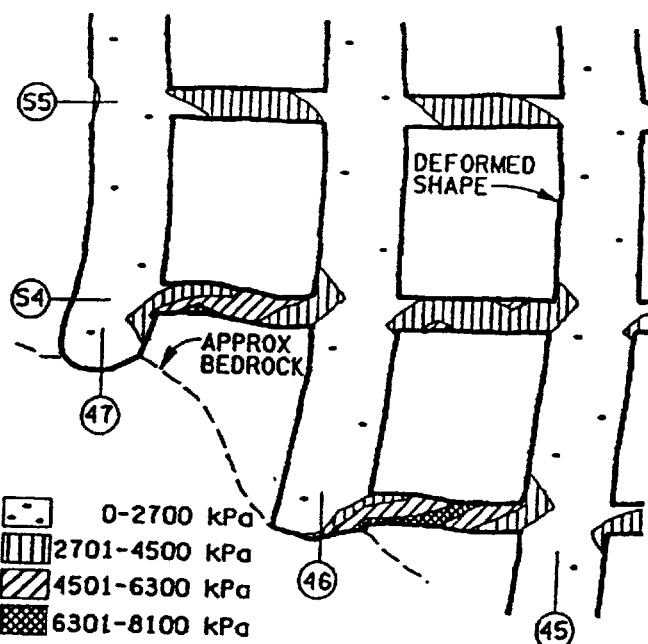


Fig. 10 — Unfactored horizontal stress contours (seismic and dead loading), portion of frame along pilaster line P5.

Base support

It is difficult to realistically represent the dam concrete/bedrock interface in a finite element model. When using a base that is assumed to be fixed or pinned to the foundation at the node points, large tensile stresses appear at the toe and heel during upstream-downstream loading. To represent more realistically the lack of tensile strength in the foundation, a single buttress finite element model was constructed with gap elements providing zero tensile capacity at the base. Resistance to upstream-downstream loading was provided by sliding friction only.

Results showed that the high tensile stresses at the heel and toe did disappear, however the magnitude of the stresses higher up in the buttress did not change significantly.

Mode combinations

The modal analysis makes available a great number of modes of little practical significance. The square root of the sum of the squares (SRSS) method was used for obtaining the total effective stresses.

For cross-valley loading of the single buttress model, it was found that the first five significant modes contributed to the majority of the total effective stresses.

The results of studies with the two dimensional frame models also showed that for cross-valley excitation only five or six modes had to be added to obtain a reasonable estimate of the total seismic response. A factor of 1.1 was applied to the set of stresses/reactions obtained to account for the effect of the unused modes.

For vertical seismic loading with the two dimensional model, it was found

that at least six modes had to be added to obtain a reasonably complete seismic response. A factor of 1.3 was applied to the set of stresses/reactions obtained to account for the effect of the unused modes. Fig. 7 shows the deflected shape for the first significant mode for each load direction of the tallest two dimensional frame model.

Haunch support

The advantages of a connection between the tops of pilasters and the upstream slab was studied. A single buttress model with the haunch only laterally supported at the buttress nose was compared with a similar model having this support in addition to lateral restraint at the top of each pilaster. A significant decrease in stresses was noted. "Strut" slabs were hence used to pro-

vide a connection between the upstream slab and the top ends of the pilasters.

Optimization of pilaster/strut layout

Pseudo-static hand calculation provided a total of nine evenly spaced pilaster/strut frames.

Optimization was achieved with iterative finite element analyses, the aim of which was to maintain generated buttress concrete tensile stresses at fairly even and acceptable levels. Seven pilaster/strut frames resulted, which were unevenly spaced (Fig. 8) along the upstream-downstream dimension of the buttresses. The uneven spacing of these frames was found to promote development of tensile stresses of similar magnitude in the unsupported zones between adjacent frames. For a cross-canyon MCE-generated ground motion, the principal tensile stresses in the original buttress concrete were limited to about 1400 kPa.

Optimization provided struts 0.7 m square and pilasters 0.7 x 0.6 m.

Structural design of remedial works

The design of all new reinforced concrete members, with the exception of strut anchors, was carried out using the limit state method. Because the MCE was considered, the seismic load factor was taken as 1.1. Design actions were attained after appropriate combination of gravity, hydrostatic, earthquake-induced, and thermal loads.

Pilasters

Pilasters provide stiffening and containment for the basically unreinforced buttress concrete. The pilasters are made to act composite with the buttress concrete and, because of their vertical orientation, are perpendicular to postulated planes of weakness at horizontal construction joints where cracking is expected to initiate.

To improve composite action with the buttresses, u-shaped "shear ties" are provided which pass through the buttresses, acting as both stirrups for the pilaster longitudinal reinforcement and as shear reinforcement at the interface. For increased ductility, hoop ties are provided around the longitudinal reinforcement in the pilasters.

The reinforcement ratio, as a percentage of the pilaster cross-sectional area, varies between 0.6 and 1.6 with an average of about 0.8. Pilaster longitudinal reinforcement was sized to with-

stand the entire elastic tensile force on the pilaster cross-section. Typical pilaster reinforcement is shown in Fig. 9.

Struts

Struts are the most critical of the load-resisting elements of the seismic rehabilitation system. In addition to cyclic tension/compression axial forces, they must also resist bending moments at the strut/pilaster joints. Struts are designed as beam-columns, supporting the composite pilaster/buttress members.

Struts are made continuous from one side of the valley to the other by passing through holes cut in the buttresses. Due to the limited installation clearance, strut reinforcement has one conventional splice per span. The reinforcement ratio, as a percentage of the strut cross-sectional area, varies between 1.9 and 3.3 with an average of 2.4. The most heavily reinforced struts are at the supports. Stress contours, unfactored for mesh density, of typical struts adjacent to supports can be seen in Fig. 10. Typical strut reinforcement details are shown in Fig. 11.

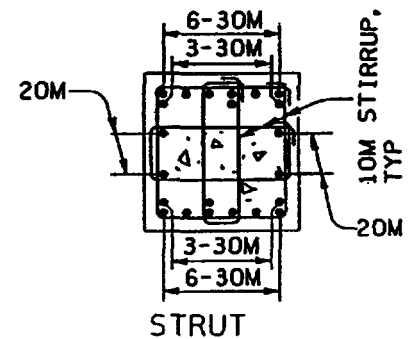
Slabs

Slabs provide the uppermost support for the new structural frame system. Cross-valley load is transferred through the slab, in diaphragm action, to dowels embedded in the underside of the existing upstream slab. The dowels are epoxy coated for corrosion resistance. One end on each slab has a sliding support to minimize thermal stresses while the other end is fixed. Typical slab reinforcing details are shown on Fig. 12.

Buttress reinforcing strips

Seismic loading in the upstream-downstream direction was found to produce high tensile stresses in the upstream and downstream edges of the buttresses. The pilaster on the downstream edge is designed to have sufficient tensile capacity to resist both the upstream-downstream and cross-canyon stresses. Along the upstream edge of ten of the highest buttresses reinforcing strips were provided to assist in the carrying of the upstream-downstream seismic stresses.

The buttress reinforcing strips are located where the calculated tensile stresses exceed 1000 kPa. Longitudinal reinforcement is designed to supplement existing reinforcement in the buttress haunches. Reinforcement dowels with hooks are provided to promote composite action with the buttress. These dowels extend into the buttress haunch to such depth as to develop the



2.82% REINFORCEMENT

Fig. 11 — Typical strut reinforcement.

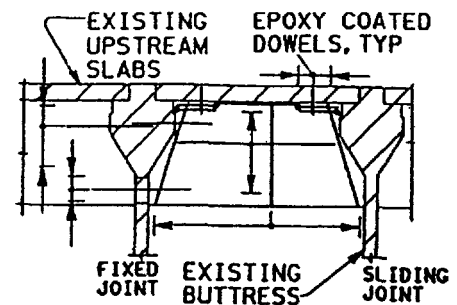


Fig. 12 — Typical plan of slab reinforcement.

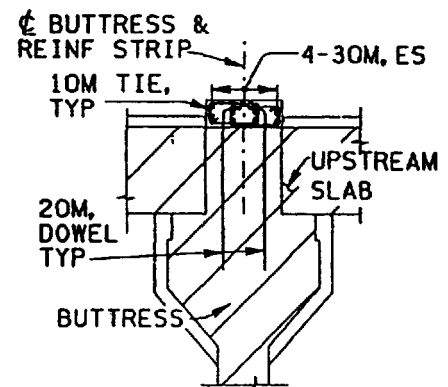


Fig. 13 — Typical buttress reinforcing strip reinforcement.

strength of the dowels at the level of the seat for the upstream slab. Ties are also provided for confinement of the longitudinal reinforcement. Typical buttress reinforcing strip details are shown on Fig. 13.

Anchors

The two dimensional frame models were used to obtain all vertical and transverse reactions at pilaster bases and strut ends for all appropriate load combinations. Global right and left bank bedrock slopes of 40 and 30 degrees respectively were used for the abutment

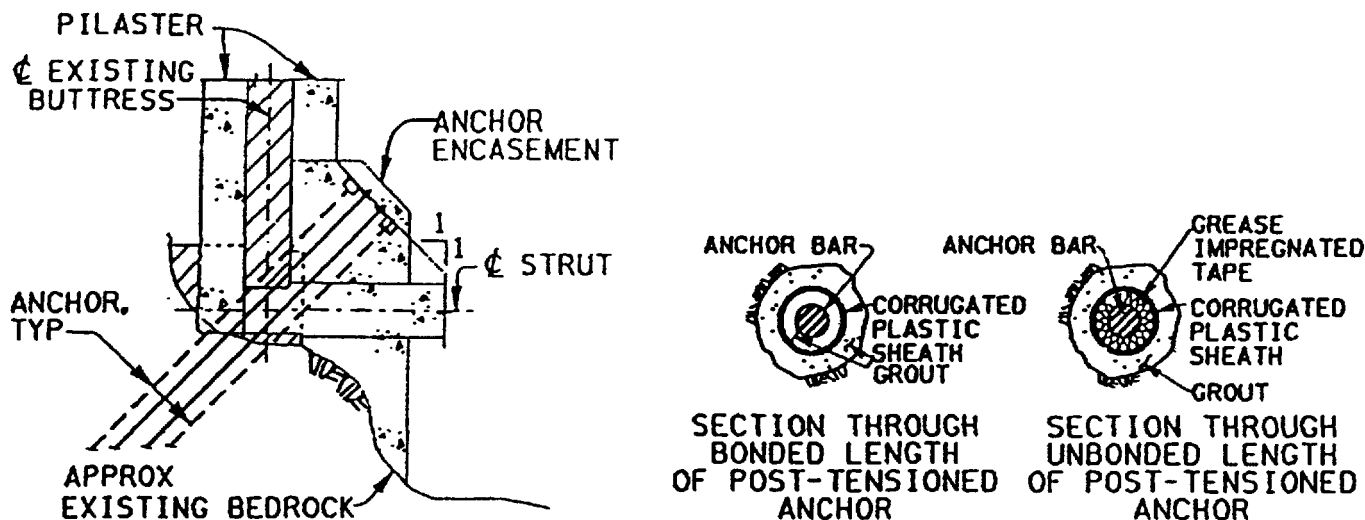


Fig. 14 — Typical post-tensioned anchor details.

surfaces. The horizontal and vertical reactions were resolved on these surfaces into normal and tangential components. Concrete/rock interface shear strength parameters, based on geological assessment of rock quality indices, were estimated to be a friction angle of 50 degrees and a cohesion of 100 kPa. Where the net driving force along the inclined surface exceeded the available resisting force, anchors were provided. It was found that all strut ends required anchors. Of all pilaster/buttress composite bases only two required anchors, at all other pilaster bases the vertical reaction, composed mostly of buttress dead weight, provided sufficient resistance to the driving force.

Geotechnical exploration showed the bedrock on the right side of the valley to be fractured, particularly in the upper 6 m. Bedrock on the left bank was typically sound and competent. For this reason, fully grouted, passive anchors, were chosen for the left bank and post-tensioned anchors for the right bank.

Although groundwater conditions were not found to be aggressive, anchors were provided with corrosion protection. Passive anchors are epoxy-coated and splice couplings are wrapped in grease-impregnated tape followed with a heat shrink plastic sleeve. Post-tensioned anchors are also epoxy-coated. Bonded lengths are premanufactured and grouted into a corrugated plastic sheath. Unbonded lengths are wrapped in grease-impregnated tape followed by a smooth plastic sleeve. Splice couplings are similar to the passive anchor design. The anchors are

single-stage grouted starting from the bottom. Typical post-tensioned anchor details are shown on Fig. 14.

Conclusions

Dynamic analyses of the Jordan River buttress dam had conclusively shown that the existing buttresses would collapse under moderate cross-canyon earthquake-induced loads. Cost estimates of the various rehabilitation alternatives had indicated that strengthening of the buttresses with a system of cast-in-place reinforced concrete pilasters and struts is cost-effective. Rigorous, iterative finite element analyses had allowed for optimization of the final design which was proven to be significantly below initial cost estimates.

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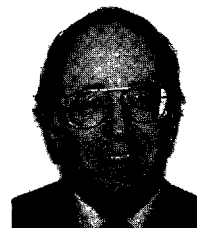
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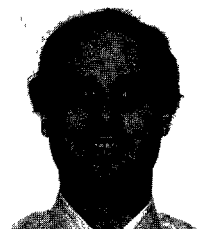
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Bay Area Rapid Transit: Concrete in the 1960s

By Bernard L. Meyers and Stephen H. Tso

The \$1.45-billion San Francisco Bay Area Rapid Transit (BART) system was the first new transit system built in the U.S. in 50 years. With preliminary planning starting in 1959, the BART system opened for service in 1972 and was completed in 1976. A joint venture of Parsons Brinkerhoff-Tudor-Bechtel (PBTB), with Bechtel as the sponsor, served as general consultants for design and construction.

The project has shown that concrete is an excellent building material for both aerial and underground structures. It has high load-carrying capabilities and an excellent ability to absorb sound and vibrations, qualities critical in a system concerned not only with the strength of the system but also with the comfort of its passengers.

Many advances have been made in the art of concrete engineering since the beginning of BART, and there has been much research on the behavior of concrete structures under dynamic loadings. Personal computers can now run all but the most demanding analytical programs. The ultimate strength design principle has replaced the working stress principle used during the design of BART. These developments, together with the advent of higher strength concretes, have enabled the engineer to design stronger and safer structures with less material than was previously possible.

Concrete also is rapidly becoming a feasible option in many situations that formerly dictated the use of steel. As an example, a precast concrete lining is so much cheaper than steel that the technical advantages of a steel lining are no longer enough to ensure its selection over the concrete alternative.

A major consideration in the design of any structure in California is its ability to withstand a major earthquake. Innovations in seismic design from the BART system are still in use today. Some of those innovations include ultimate strength design and the design of underground structures to

absorb ground deformations rather than to resist an applied lateral load.

The system is 75 miles (120 km) long and connects 14 cities in San Francisco, Alameda, and Contra Costa counties. One-third of the system is underground, consisting of 17 miles (27 km) of subways, the 3.6-mile (5.8 km) Trans-Bay Tube, and 4 miles (6.4 km) of rock tunnels. Another third consists of 25 miles (40 km) of aerial structures, with the remaining third being 25 miles (40 km) of at-grade construction. There are 34 stations and 3 maintenance and storage yards servicing the 10-car trains in the BART system.

Aerial structures Standard structures

Aerial structures make up about 25 miles (40 km) of the BART system, with standard design structures constituting more than 90 percent of this total. Standard designs were prepared for girders, pier beams, columns, foundations, and abutments.

Girder design — Designs were prepared for three types of construction: structural steel, conventionally reinforced concrete, and prestressed concrete. This allowed contractors to choose the most economical option for each particular site. Spans reached up to 40 ft (12 m) for reinforced concrete girders and 96 ft (29 m) for both steel girders and precast post-tensioned girders. The precast post-tensioned concrete option using 6000 psi concrete was selected by every contractor.

Standard concrete girders were 4 ft (1.2 m) deep trapezoidal boxes with 11 ft-8 in. (3.6 m) wide decks. To enhance appearance, the deck edges were cantilevered from the sides of the boxes, which were set at a 3-to-1 slope. Each standard girder carried one track. Girder ends were notched to allow the pier beam to be partly contained within the girder depth. This scheme provided a higher vertical clearance and a sleeker profile compared to a

girder without notches. Variable depths were used on the right and left sides of the box to provide the required superelevation. A girder cross section is shown on Fig. 1.

An innovative feature was the use of continuous welded rails with direct fixation fasteners. The rails were bolted directly to the girder decks and second stage concrete was used to accurately install the rails. This scheme eliminated the need for cross ties and ballast, and saved millions of dollars. The continuous welded rails also provided additional seismic resistance in the longitudinal direction.

Standard design pier beam, foundations, and columns — Elastomeric bearing pads supported the girders onto the pier beams. A 2 ft (0.6 m) wide closure placement joined the girders after they were erected. At the fixed end of each girder, rebar extended from the precast piece into this placement. At the expansion end, a 5 in. (127 mm) square steel tube extended from the precast piece into a teflon-lined steel sleeve within the closure placement. Two 5 in. (127 mm) round steel pins extended from the top of the pier beam into the closure placement. The girder end connection detail is shown in Fig. 2.

Fig. 3 shows a typical column and pier beam. Pier beams and columns were either 5 or 6 ft (1.5 to 1.8 m) wide. The pier beam was 22 ft (6.7 m) long to accommodate a track spacing of 14 ft (4.3 m). The maximum depth was 5 ft (1.5 m) and tapered up at a 4-to-1 slope from the column. Standard columns were hexagonal and either 5 or 6 ft (1.5 or 1.8 m) wide across the flats. Longitudinal reinforcing steel consisted of #18 bars confined within a $\frac{5}{8}$ in. (16 mm) diameter spiral at a 3 in. (76 mm) pitch. The spiral extended from about 3 ft (0.9 m) inside the beam down to about 2½ ft (0.8 m) inside the footing to provide the necessary confinement for structures in a high seismic zone. The maximum

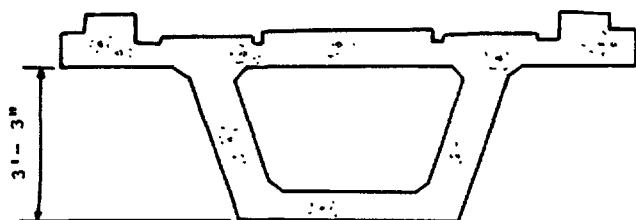


Fig. 1 — Typical aerial girder section; rebar not shown.

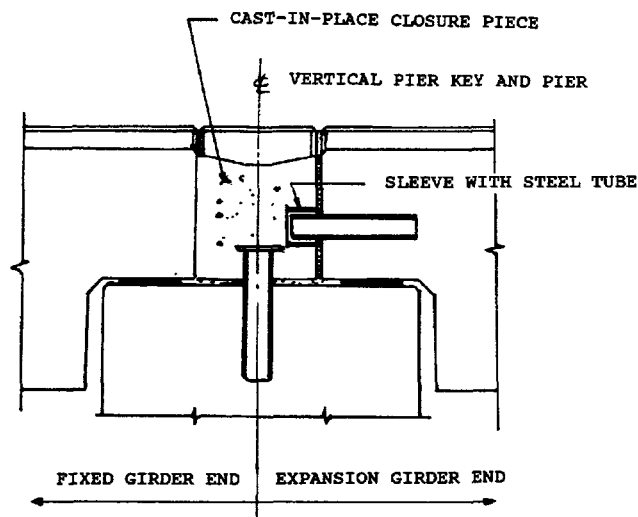


Fig. 2 — typical girder end details; rebar not shown.

umn height was 45 ft (13.7 m) between the top of the rail and the bottom of the footing, or about 34 ft (10.4 m) clear height between the girder soffit and the top of the footing. The reinforcing steel strength was 40 ksi (275 MPa) yield and the concrete compressive strength was 4 ksi (28 MPa) at 28 days.

Standard footing designs used either spread footings or 30, 45, or 60 ton (27, 41, or 54 Mg) capacity piles. Spread footing designs used allowable soil bearing pressures of 2, 3, 4, or 5 tons per square foot (190, 290, 380, or 480 kPa) and ranged in size from 18 to 28 ft (5.5 to 8.5 m) square. The footings on piles ranged from 14 to 24 ft (4.3 to 7.3 m) square. The concrete strength used in footings was 3000 psi (21 MPa).

Typical pile types used were steel HP, concrete-filled steel shell, precast pretensioned concrete, and timber. The pile type was chosen to suit specific site conditions. To resist lateral loads, exterior piles were battered at a 4:1 slope and interior piles were battered at 8:1. Generally, pile foundations were used instead of spread footings even in areas of good soils. This was because the criterion limiting differential settlement between piers to an

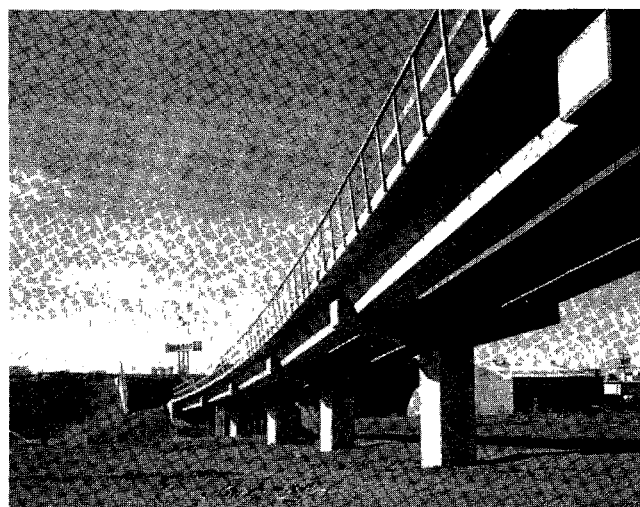
amount equal to the length tributary to the pier divided by 2400 was more easily met with a pile foundation.

were used where the standard structures were not suitable. These special designs included structural steel girders, cast-in-place post-tensioned concrete girders, C-bents, and double column straddle bents.

Seismic design requirements

Seismic design criteria permitted a one-third increase in the basic allowable stresses for minor earthquakes with ground acceleration of 0.10g. In addition, an ultimate strength design criteria was adopted to resist loads from a major earthquake. The criteria assumed a certain amount of repairable damage, but no loss of function or collapse. Aerial structures were required to withstand a specified horizontal deformation without straining the column reinforcing steel past twice its yield strain.

During the 1989 Loma Prieta Earthquake, sensors recorded a free field peak ground acceleration of 0.15g horizontally and 0.08g vertically. Maximum accelerations on the girder deck reached 0.60g in the transverse direction and 0.26g longitudinally. The resulting forces were not high enough to cause any damage.



Typical aerial structure.

amount equal to the length tributary to the pier divided by 2400 was more easily met with a pile foundation.

Special aerial structures

Special designs

Aerial stations

Both center and side platform stations were used, although center platforms were preferred because one set of stairs and escalators could serve both inbound and outbound passengers. The structural system consists of double column bents acting as moment frames in the lateral direction and as cantilevering columns in the longitudinal direction. Bents were spaced at 70 ft (21 m) to match BART car lengths. Standard girders spanned between bents and separate framing supported the passenger concourse and platforms. The total platform length was 700 ft (210 m) to match a 10-car train.

Cut-and-cover structures

There are eight cut-and-cover lines and ten cut-and-cover stations in the BART system. The typical soil profile consisted of varying layers of dense sands, clayey sands, and sandy clay, often underlaid by stiff, silty clays. The water table was high but the soil was usually dewatered during construction. Earth cover over structures averaged from 8 to 9 ft (2.4 to 2.7 m) at stations and 20 ft (6.1 m) at line sections.

Seismic design considerations

Most of the BART subway structures were constructed in dense or silty sand, which is stiff compared to the structure. In such an environment the structure will simply conform to soil distortions; thus, the criterion for design was to absorb the imposed deformation rather than to resist applied forces.

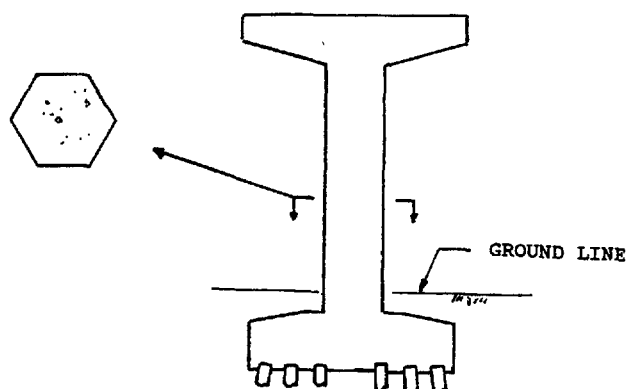


Fig. 3 — Typical column and pier beam; rebar not shown.

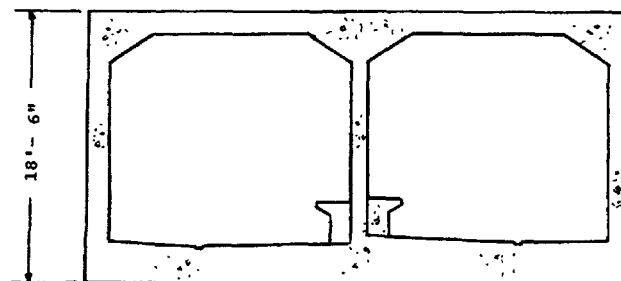


Fig. 4 — Typical two-cell structure; rebar not shown.

Structural systems

The structural system for most cut-and-cover lines and stations was the multicelled reinforced concrete box, with walls and slabs acting one way in the transverse direction to form a continuous rigid frame to resist both the vertical and lateral loads (see Fig. 4). The box-type construction offered high resistance to shear and to uplift forces from a high groundwater table.

In stations, the reinforced concrete box with concrete or structural steel girders and columns was designed and built with full or partial continuity in the transverse direction. Longitudinal girders and interior columns were used to open up the station interior. Where concrete girders and columns were used, advantage was taken of the slab-band type of design, i.e., using wide bands at the columns instead of deep girders to keep construction depth shallow. Alternatively, using structural steel columns and longitudinal girders instead of concrete columns and wide girders had the advantage of reducing the column sizes and making the platform and mezzanine areas more open.

Construction methods

Construction methods along the BART subway alignment depended on many factors including site and environmental conditions and line and station configurations. Soldier piles and lagging were used for subway structures with favorable soil conditions and with underpinning of foundations when the excavation line was close to existing buildings. Sheet piling was used where dewatering resulted in detrimental settlements or where the or-

ganic clay extended appreciably below the bottom of the excavation. Where organic clay deposits were present and the settlement of adjacent structures needed to be minimized, the rigid bulkhead method was used.

Tunnels

In general, single-track tunnels were bored between stations where the lines went between busy downtown streets. With the exception of the Berkeley Hills and Fairmont tunnels, which pass through rock formations, all tunnelling was done in alluvial ground, with much of it below the water table.

Rock tunnel

Seismic design consideration —

The Berkeley Hills and the Fairmont Hill tunnels are the only rock tunnels in the BART system. The Fairmont Hill Tunnel is approximately 4000 ft (1220 m) long, while the Berkeley Hills Tunnel consists of two single-track bores 16,000 ft (4877 m) long, 100 ft (30 m) apart with up to 600 feet (183 m) of rock cover.

The Berkeley Hills Tunnel is the only structure in the system that crosses an active earthquake fault — the Hayward Fault — and the primary concern in its seismic design was movement and distortion of the fault. To accommodate tectonic deformation and to permit rapid repair and adjustment of track after a major earthquake, the Berkeley Hills Tunnel was oversized from BART's typical subway tunnel diameter of 16.5 to 17.5 ft (5.0 to 5.3 m). During excavation, the tunnel was lined throughout with a primary lining of steel ribs and a

secondary lining of cast-in-place concrete. A post-Loma Prieta Earthquake inspection revealed no damage to the BART tunnels and that no realignment was necessary.

Soft ground tunnel (tunnel linings)

Design criteria — One of the major engineering accomplishments of the BART Project was the development of design criteria and construction techniques that made shallow tunnels in soft ground both feasible and economical. Soft ground tunneling is affected by the amount of ground deformation, which depends on the physical properties of the ground, how well the tunnel face is supported, and how soon and completely the annular space behind the shield tail is filled with grout.

Experience with existing soft ground tunnels of similar size in similar soil conditions indicated that the deformation of the lining after it has been grouted into contact with the surrounding soil generally can be kept to 1 in (25 mm). The design criteria of the BART tunnel lining was based on a capacity to accept this deformation, in combination with a uniform radial pressure equal to the overburden weight plus the hydrostatic pressure, without exceeding normal working stresses.

Design alternatives — Two possible lining systems were examined for both performance and economy:

System I — A segmented lining of cast iron, fabricated steel, or precast concrete

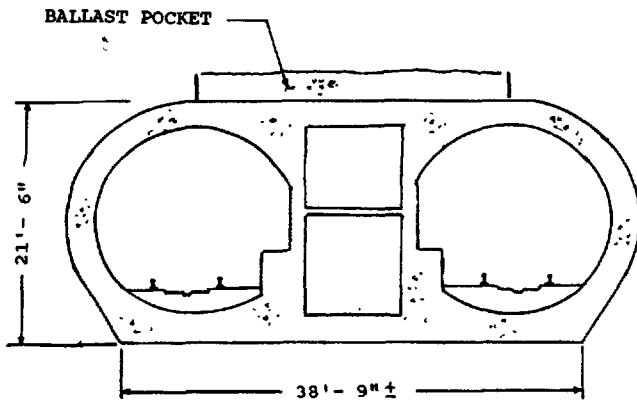
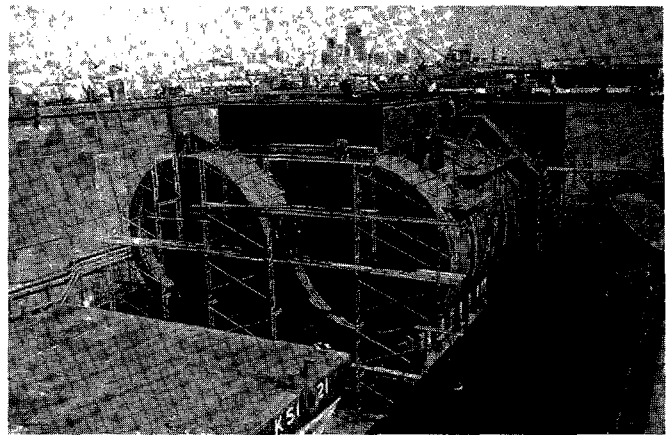


Fig. 5 — Typical section of Trans-Bay Tube; rebar not shown.



Tunnel construction.

- System II — Cast-in-place lining (with or without initial temporary lining)

Evaluation of the two systems for performance and cost gave the following results:

- The ground settlements due to the installation of the tunnels would be one-third less for System I than for System II.
- System I required less field control to achieve a sound structure. The bolted prefabricated steel segment construction produced a more watertight tunnel than the precast concrete construction.
- System I could be built faster than System II.
- System II generally has better impact and fire resistance qualities than System I.
- System I (with either cast iron or fabricated steel segments) was 7 to 20 percent cheaper than System II at 1970 prices. (Since then, the price advantage has shifted toward the precast concrete segmented lining.)

Metallic linings were chosen as superior to precast concrete segmented lining because of less ground settlement during construction, better durability, and faster construction. Further evaluation of the relative in-place costs led to the adoption of fabricated structural steel segmented liners for all BART subway tunnels.

Trans-Bay Tube

The 3.6 mile (5.8 km) Trans-Bay Tube is the longest and deepest underwater crossing of its type in the world. The twin-track tube connecting San Francisco with the East Bay region was the most challenging design problem in the entire project. Responsibility for developing and designing the

tube was given to Parsons Brinkerhoff, Quade & Douglas.

The tube is binocular in cross section (see Fig. 5) containing two track compartments and a central area for utilities and an inspection gallery. The tube is founded on a course of gravel in a dredged trench on the bay bottom and changes alignment both horizontally and vertically in the 3.6-mile length between ventilation structures, dipping to about 135 ft (41 m) below sea level. Generally, the top of the tube lies at least 5 ft (1.5 m) below the bay bottom. Grades are limited to 3 percent, with the minimum grade for drainage being 0.3 percent.

Design concept and earthquake consideration

Though the tube crosses no major active faults, it is possible that the tube could be subjected to ground deformation caused by earthquakes originating along the San Andreas and Hayward Faults. Consequently, a primary design consideration was to locate the tube in a bed of soft alluvial material that would absorb much of the seismic force instead of transmitting it to the structure.

The other major concern was the possibility of shear failure of the deep stratum of relatively soft silty clays, known as "bay mud," as a result of ground shaking. To investigate this matter, soil samples from the bay bottom were subjected to dynamic laboratory tests specially devised to simulate earthquake effects. The results showed that the shear strength of the bay mud increased rapidly with depth, reaching up to 800 to 1000 psf (38 to 48 kPa) at 50 ft (15 m) below bay bottom. At the surface, the shear strength of the bay mud was about 200 psf (9.6 kPa). It

was concluded that any shear failure would occur in loose material not deep enough to damage the tube.

Liquefaction phenomena also were investigated and found to be possible only under vibrations far in excess of those associated with earthquakes.

As a final check, a model simulating a 300 ft (91 m) long tube section in layered soil similar to the bay bottom was tested at the large shake table at the University of Mexico. The tests confirmed the validity of the analytical procedures used in predicting tube movements during an earthquake. After the 1989 Loma Prieta earthquake, a post-earthquake inspection of the Trans-Bay Tube revealed no structural damage. Despite an observed $\frac{3}{4}$ in. (19 mm) movement in the longitudinal direction, no trackwork realignments were necessary.

Design features

The Trans-Bay Tube is a composite structure consisting of a $\frac{3}{8}$ in. (10 mm) thick exterior steel shell and a 2 ft (0.6 m) thick reinforced concrete lining inside the tube. The shell and lining creates a ductile waterproof structure capable of withstanding displacements due to earthquakes. The structural steel shell is stiffened transversely by 8 in. (203 mm) structural steel tees spaced at 6 ft (1.8 m) centers. Elastic joints of neoprene rubber are used to absorb differential deformation. These joints permit about 6 in. (152 mm) of longitudinal movement, plus associated rotational and transverse movement.

Construction

Trans-Bay Contractors (a joint venture of Kiewit, Raymond, Tidewater & Healy-Tibbitts) was the contractor for

the tube construction. Tube sections were floated to the site and sunk into position with 500 tons (455 Mg) of ballast placed in pockets on top of the sections. The tube was placed in a dredged trench beneath the bay bottom and covered with protective layers of gravel, sand, and a 2 to 3 ft (0.6 to 0.9 m) thick armor blanket of stone. A 2 ft (0.6 m) bed of $\frac{3}{8}$ to 3 in. (10 to 76 mm) rock was placed as a foundation course in the bottom of the trench.

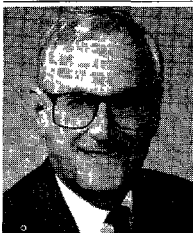
Epilogue

From its beginning twenty years ago, BART has delivered on its promise of safe, reliable, and economical transportation within the Bay Area. BART remained in operation during the Loma Prieta Earthquake, thereby maintaining a vital link throughout the local economy. On October 1991, BART embarked on an ambitious program to add about 32 miles (51 km) of trackway and 10 to 12 new stations to the existing system. Due to its many advantages, concrete will once again be the material of choice, this time aided by the new advances in concrete engineering and by the analytical capabilities of the personal computer.

Acknowledgments

The authors wish to thank the many colleagues who contributed to this article, especially Carl Goeppfert, Peter Frobenius, Dave Juday, C.C. Wang, Richard Tso, and Richard Leong.

Selected for reader interest by the editors.



his PhD from Cornell University.

Bernard L. Meyers is a senior vice president of Bechtel Corporation and manager of the Bechtel Nuclear Business Line. He received

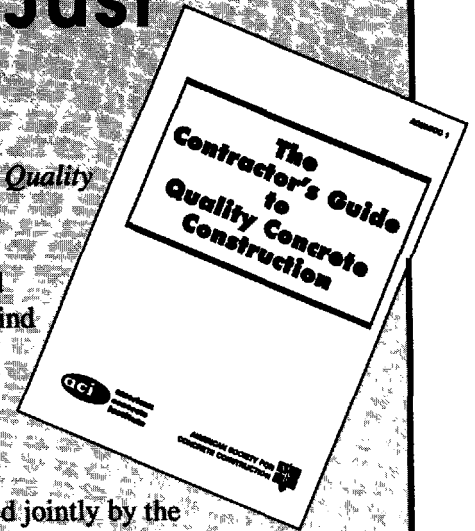


been responsible for the structural design and management of many projects worldwide, including the Bay Area Rapid Transit System.

Stephen H. Tso, ACI member, is chief civil structural engineer in Bechtel Corporation's San Francisco Regional Office and has

Achieving Quality Concrete Just Got Easier

... with *The Contractor's Guide to Quality Concrete Construction*, a new book written by and for those who order, form, place, consolidate, finish, and cure concrete. Designers also will find the practical approach of *The Contractor's Guide to Quality Concrete* valuable in developing their project specifications.



The Contractor's Guide is published jointly by the American Society for Concrete Construction and the American Concrete Institute, and has been reviewed by ACI Committee E703, Concrete Construction Practices, in accordance with the ACI Educational Activities Committee Document Review Policy, and by the ASCC Board of Directors.

The purpose of the *Contractor's Guide* is to provide insight into proven construction practices which, when properly used, will produce quality concrete construction. It can be used as a training manual for contractor personnel as well as a basic reference manual for field and office. Contents include organizing for quality; the concrete mix; concrete specifications; foundations; formwork; reinforcement in structures; joints and embedments in structures; joints and reinforcement for slabs-on-grade; preparing for concreting; concrete placement and finishing; common field problems; and safety.

Specification changes often can improve performance, reduce costs without adversely affecting quality, and relieve undue hardship for the contractor, and should be given serious consideration by designers. The best time to do this is at the bidding stage. Contractors will find the text of this book helpful in support of such changes.

Throughout this book are practical contractor tips that will be especially helpful in preparing for a project and as a guide during construction. *The Contractor's Guide to Quality Concrete* is a must-have document. Contact the ACI Member/Customer Services Department at (313) 532-2600 to order yours today, or FAX your order to (313) 533-4747.

1992, 107 pages, soft cover

Precast Concrete Connection Details For All Seismic Zones

by C. E. Warnes

The analysis of reinforced concrete structures is generally based on the assumption that all of the separate elements behave as a unit. Of course, the actual structure must be built piece by piece due to construction limitations. Nevertheless, one of the principal elements of the art of design is to produce monolithic behavior in the finished structure.¹

Properly located construction joints, whether in precast or cast-in-place structures, will ensure transmission of compressive forces in concrete and reinforcing steel. All tensile forces required to cross the construction joint must be carried by the reinforcement.

Just as it is impossible to cast all concrete in one monolithic, continuous operation, it is also impossible to provide full length continuous reinforcement throughout any large structure. Splices

of reinforcement are unavoidable. Properly designed splices are a key element in any well executed design. Building codes limit the methods of joining reinforcing bars to lapping (except #14 and #18 bars), welding, and the use of mechanical connections. Many splice situations are unique and require custom-made solutions.

The location of construction joints and splices of reinforcement in precast concrete are controlled by the practicalities of manufacturing, transporting, and erecting precast elements. As in cast-in-place concrete, the precast structure must be built piece by piece. The difference between the two forms of reinforced concrete occurs primarily in the methods used to join the elements.

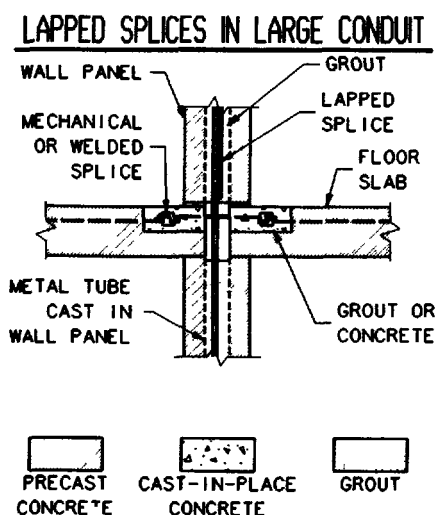
The conventional approach used in North America in the past for connecting precast structural members has relied principally upon the use of "jointed" elements. The joints between precast concrete structural units generally consist of weldments of steel plates, bars, tubes, angles, studs, reinforcing bars, and bolts in various combinations. The continuity of reinforcing bars

through a joint is usually interrupted by a fabricated assembly of some type.

In Japan and New Zealand especially and, to an increasing degree in North America, precast concrete structures are being constructed that attempt to behave like cast-in-place monolithic structures.^{2,3} This "cast-in-place equivalent" or "emulation" approach designs precast concrete structures in such a manner that their stress response characteristics closely duplicate those of cast-in-place structures.⁴

Building code requirements for cast-in-place concrete

The three U. S. model building codes all utilize the ACI Building Code for the concrete sections of their requirements. The Standard Building Code (Southern Building Code Congress International, Inc.) and the BOCA National Building Code (Building Officials and Code Administrators International, Inc.) both adopt the ACI 318 Code by reference.



SECTION AT JOINT

Fig. 1 — Lapped bar splices in grout filled conduit which are extended full height through the structural element.

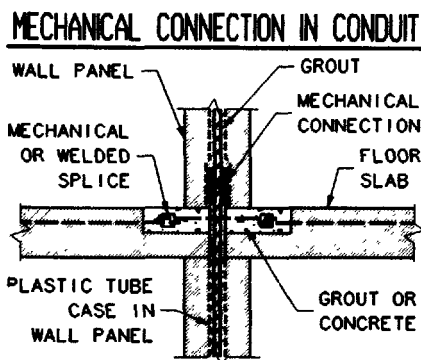
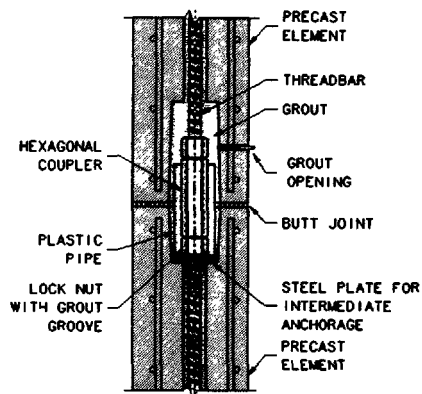


Fig. 2a — Special bars with rolled threads are joined with a threaded female mechanical connection and the system is grouted.



MECHANICAL CONNECTION USING "THREADBAR"

Fig. 2b — Detail of "threadbar" system.