# Design curves and charts for static and seismic analysis of plain concrete quay walls

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There are a lot of factors affecting the performance of concrete blocks quay walls. Among those factors; depth of quay wall, filling material properties, bearing stratum characteristics, gross ship weight and pulling force, shape and arrangement of plain concrete blocks and seismicity. Although seismicity varies regionally, earthquake disasters have repeatedly occurred not only in the seismically active regions in the world but also in areas within low seismicity regions. Mitigating the outcome of earthquake disasters is a matter of worldwide interest. This research focuses on the stability of concrete block quay wall under static and seismic loads. To make it possible for design engineers to come into a possible, economic and safe design, a complete set of design charts covering a wide rang of parameters are presented.

حائط الرصيف من البلوكات الخرسانية من الحوائط الأكثر انتشاراً في الموانئ. هناك عوامل كثيرة تتحكم في اتزان ذلك الحائط من بينها القوى الناتجة من الزلازل. وبالرغم من تباين مناطق قوة الزلازل إلا إن تأثيرها يجب أن يؤخذ في الحسبان دائما نظراً لقوة تدميرها. يركز هذا البحث في تحقيق اتزان الحائط تحت تأثير الأحمال الاستاتيكة والديناميكية باستخدام الطرق المبسطة وبرنامج حاسب آلي تم كتابته خصيصاً لهذا الموضوع. ولأهمية هذا النوع من الحوائط تم تقديم مجموعه متكاملة من منحنيات وأشكال التصميم لمدى واسع من المدخلات حتى يمكن للمهندسين والخبراء في هذا المجال الاستعانة بمعلومات موثقه من فهم وتحليل وتصميم هذا النوع الشائع من الحوائط بطريقه افضل.

**Keywords**: Quay walls, Gravity walls, Harbors, Berths and docks design, Seismic analysis

## 1. Introduction

In order to mitigate hazards and losses due to earthquakes, seismic design methodologies have been developed and implemented in design practice in many regions since the early twentieth century, often in the form of codes and standards. Most of these methodologies are based on a force-balance approach, in which structures are designed to resist a prescribed level of seismic force specified as a fraction of gravity. These methodologies have contributed to the acceptable seismic performance of port structures, particularly when the earthquake motions are more or less within the prescribed design level. A simplified method has been introduced to imply for seismic forces.

QWD is a computer program, which has been implemented using visual basic.net and .net framework technology to design a gravity quay wall and check an existing one under static and dynamic effects. A parametric study has been done and results are introduced in the form of design charts.

## 2. Simplified analysis of plain concrete quay walls under static and seismic loads

In addition to the traditional static forces, a set of seismic parameters should be looked at. The method of calculation in this case is based on the Mononobe-Okabe (M-O) method [1], which is a direct extension of the static Coulomb theory to pseudo-static conditions.

### 2.1. Forces acting on the wall

Two types of forces act on the gravity retaining quay wall, namely stability forces and failure forces.

i. *Stability forces.* The two vertical stability forces are: the weight of soil over the projection part of the block and the own weight of blocks.

- *Weight of soil* = $\sum W_1$; *where:*

$W_1$ = the weight of the soil resting on top of the projection part of the block (in tons), acting at the center of gravity (c.g.) of each part.

- *Weight of blocks*= $\sum W_2$; *where:*

$W_2$= Own weight of blocks (in tons), acting at the center of gravity (c.g.) of each block.

- *Total Stability forces acing on the wall:*

Total Stability forces acing on the wall per meter length (in tons).

$\sum F_{stability}$ = $W_1$+ $W_2$+ vertical crane load (if exists).

*ii. Failure forces.* There are four horizontal forces acting as seismic failure forces; Lateral earth Pressures, the pull bollard, the live load and the horizontal crane load effect.

- *Pull Bollard:*

Pull bollard (in tons), acting at 0.4 m above the top of quay wall level.

- *Live load:*

Live load (in ton/m$^2$) depends on the type of the quay wall.

$$P_{LL} = \frac{1}{2} * L.L. * K_A * H^2 .$$

Acting at 0.5 *H*.

- *Horizontal crane load:* If it is existed, Horizontal Crane load (ton) = 1/7 Vertical Crane load, [1], acting at the top of head.

- *The hydrodynamic water pressure:* The hydrodynamic water pressure force for the free water within the submerged backfill, Pwd, is given by the Westergaard relationship [2].

$$P_{wd} = \frac{7}{12} * k_h * \gamma_{w1} H^2 .$$

And acts at 0.4 *H* above the base of the wall,

Where:

$k_h$ is the horizontal seismic coefficient as a friction of *g*.

$$\gamma_{w1} = \gamma_w + \gamma_{sub} * ru.$$

$\gamma_w$ is the unit weight of water = 1 (ton/m$^3$), and;

*ru* is the excess pore water pressure ratio.

- *Seismic lateral earth Pressures (dry layer):* Seismic lateral earth pressures dry layer is the area of backfill pressure of dry layer acting on the wall per meter (ton) calculated by The Mononobe-Okabe theory [1].

$$P_{AE(dry)} = \frac{1}{2} K_{AE(dry)} \gamma H^2 (1 - K_V) .$$

Acting at,

$$Y_{PAE} = \frac{P_A * Y_{PA} + \Delta P_{AE} * 0.6(H * h_s)}{P_{AE}} .$$

Where:

$P_A$ is the static lateral earth pressures-dry layer and $h_s$ = Live load/ $\gamma$ ,

$\gamma$ is the dry unit weight of soil (ton / m$^3$), and

$k_v$ is the vertical seismic coefficient as a friction of *g*.

The seismic active earth pressure coefficient, $K_{AE\,(dry)}$, is given by:

$$K_{AE(dry)} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi)\left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi)\cos(\beta - \theta)}}\right]^2} .$$

Where:

$\theta$ is the inclination of wall (degree),

$\beta$ is the slop angle of quay wall level (degree),

$\delta$ is the interface Friction angle of soil (degree),

$\phi$ is the angle of internal friction of soil (degree), and;

$\phi - \beta \geq \psi$ and $\psi = \tan^{-1}[k_h / (1 - k_v)]$.

- *Seismic lateral earth pressures (submerged layer):* Seismic lateral earth pressures-submerged layer is the area of submerged backfill pressure with excess pore pressures acting on the wall per meter (ton). The pore water pressures may increase above their steady state values in response to the shear strains induced within the saturated portion of the backfill during earthquake shaking, as discussed in [3 - 13].

$$P_{AE(sub)} = \frac{1}{2} K_{AE(sub)} \gamma H^2 (1 - K_V) .$$

Acts at

$$Y_{PAE} = \frac{P_A * Y_{PA} + \Delta P_{AE} * 0.6(H * h_s)}{P_{AE}}.$$

In which:
$P_A$ is the static lateral earth pressures-submerged layer and $h_s$ =Live load/ $\gamma_d$,

$\gamma = \gamma_{sub} * (1 - ru)$.

Where:
$ru$ is the excess pore water pressure ratio.
The seismic active earth pressure coefficient, $K_{AE\ (sub)}$, is given by:

$K_{AE(sub)}$ =

$$\frac{\cos^2(\phi - \theta - \psi^*)}{\cos\psi^* \cos^2\theta \cos(\delta + \theta + \psi^*)\left[1 + \sqrt{\frac{\sin(\delta + \phi)\sin(\phi - \beta - \psi^*)}{\cos(\delta + \theta + \psi^*)\cos(\beta - \theta)}}\right]^2}.$$

Where:
$\phi$ is the average angle of internal friction of soil and rock (degree).
$\phi_{av} = \phi_r + 2/3\ (\phi_r - \phi_s)$ (Ibrahim. A. Ebeido, [14]).

Where:
$\phi_r$ is the angel of internal friction of rock.
$\phi_s$ is the angel of internal friction of soil.

$\phi_{av} - \beta \geq \psi^*$, in which

$$\psi^* = \tan^{-1}\left[\frac{\gamma_d k_{h1}}{\gamma_{sub}(1 - k_v)}\right],$$

and;

$$K_{h1} = \left[\frac{\gamma_d}{\gamma}\right] K_h .$$

*- Total seismic failure forces acting on the block:*
$\sum F_{failure}$= ($P_{AE\ (dry)}$ + Pull Bollard + Vertical Crane load) * cos $\delta$

Where:
$P_{AE\ (dry)}$ is the area of seismic lateral earth pressures and live load.

$$P_{AE(dry)} = K_{AE(dry)}\left(1 + \frac{2 * liveload}{\gamma_d * H_1}\right) * \frac{1}{2}(\gamma_d * (1 - k_v)) * H_1^2 .$$

$P_{AE(sub)}$ is the area of seismic lateral earth pressures and live load.

$$P_{AE(sub)} = K_{AE(sub)}\left(1 + \frac{2 * liveload}{\gamma_{se3} * H_2}\right) * \frac{1}{2}(\gamma_{se3} * (1 - K_V)) * H_2^2 .$$

In which: $\gamma_{se3}$ is the effective unite weight of submerged soil due to seismic lateral earth pressures

*2.2. Safety against sliding*

The factor of safety against sliding is expressed as:

$$F.O.S._{slding} = \frac{\sum F_{stability}}{\mu * \sum F_{failure}} \geq 1.5.$$

Where $\mu$ = friction coefficient between blocks.

*2.3. Safety against over turning moment*

The safety factor against over turning moment is expressed as:

$$F.O.S._{overturning} = \frac{\sum M_{stability}}{\sum M_{failure}} \geq 1.5 .$$

*2.4. Stresses*

$$e = \frac{blocklength}{2} - \left(\frac{\sum M_{stability} - \sum M_{failure}}{\sum F_{stability}}\right),$$

$$f_1 = \frac{-\sum F_{stability}}{blocklength}\left(1 + \frac{6 * e}{blocklength}\right),$$

$$f_2 = \frac{-\sum F_{stability}}{blocklength}\left(1 - \frac{6 * e}{blocklength}\right).$$

For last block the following conditions should be satisfied:
$f_1 \leq$ bearing capacity, $f_1$, $f_2$ (–ve sign) compression and $2 * f_2 > f_1$. If the conditions are not satisfied, we have to adjust the dimension of the bottom block and the one directly above the bottom block. If this is not enough, we may enlarge the third block from the base.

## 3. Effect of seismic parameters on the behavior of the quay wall.

Fig. 1 shows the different forces acting on the plain concrete quay walls.

Table 1 shows the parameters implemented in the present analysis. Note that; Horizontal seismic coefficient as a friction of g ($kh$) = 0.1, Vertical seismic coefficient as a friction of g ($kv$) = 0 and Excess pore water pressure ratio ($ru$) = 0.1.

Fig. 2 shows the relation ship between lateral pressure force acting on the bottom block and the angle of internal friction for seismic case.

Water level also affects the design. In the static case it is expected that the pressures increases with the decrease of water level while in the seismic case, the worst case is achieved when water table rises. This is due to the pore water pressure.

Horizontal seismic coefficient has a great effect on pressure as it can be seen from fig. 3. It is not the case for the vertical one fig. 4. The vertical coefficient can be disregarded.

Pore water pressure seems to have a consistence effect on lateral pressure for wide range of water pressure, but large magnitude of excess water pressure cusses a dramatic rise up in lateral pressured acting on the wall, fig. 5.

Gun pulling force is usually taken (2 t/m) when the total pulling force is increased; the number of mooring guns is increased too.

## 4. Design charts

The developed computer program has made it possible to produce a numerous number of design charts covering a wide range of quay wall parameters.

As it is well known, the design of plain concrete blocks focuses on predicting the length of each block including the head (cast

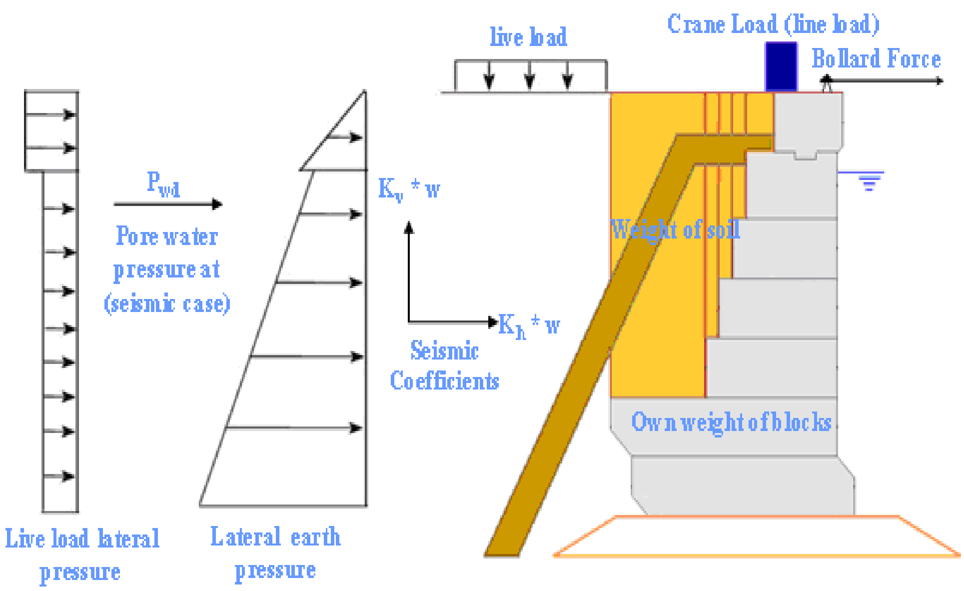


Fig. 1. Wall configuration and static and seismic acting forces.

in place crown). The designer normally assumes the height of each block and usually it is taken between 1.5 to 2.5 meters. The width of each block is determined according to the capacity of the lifting crane.

The design charts are divided into six groups. Parameters of these groups are shown in tables 2 and 3 for static and seismic cases, respectively. In all graphs the height of the first block= (draft + clearance + tidal range)-(height of block*(number of block-1)). All the charts focus on finding the number of blocks and the length of each block. Width can be calculated according to the crane capability.

Table 1
Parameters of the quay wall model

| Parameter | Chosen value |
|---|---|
| Draft (m) | 12 |
| Clearance (m) | 1 |
| Tidal range (m) | 1.5 |
| Distance between upper surface of head and H.W.L ( | 1.5 |
| Lower surface level of the head (m) | +0.75 |
| Height of each block (m) | 2.25 |
| Height of first block (m) | 2.5 |
| Horizontal seismic coefficient as a friction of g (kh) | 0.1 |
| Vertical seismic coefficient as a friction of g (kv) | 0 |
| Excess pore water pressure ratio (ru) | 0.1 |
| Saturated unit weight of concrete (ton/m$^3$) | 2.2 |
| Buoyant unit weight of concrete (ton/m$^3$) | 1.2 |
| Friction coefficient (concrete/ concrete) | 0.4 |
| Friction coefficient (concrete/ rock) | 0.5 |
| Angle of internal friction of soil(degree) | 30 |
| Angle of internal friction of back fill (degree) | 45 |
| Average dry unit weight of soil (ton/m$^3$) | 1.85 |
| Average buoyant unit weight of soil (ton/m$^3$) | 0.9 |
| Bearing capacity of soil (ton/m$^2$) | 25 |
| Buoyant unit weight of rock (ton/m$^2$) | 1 |
| Angle of interface friction (soil/structure) (degree) | 0 |
| Pulling force on bollard (ton/m) | 2 |
| Live load (ton/m2) | 2 |

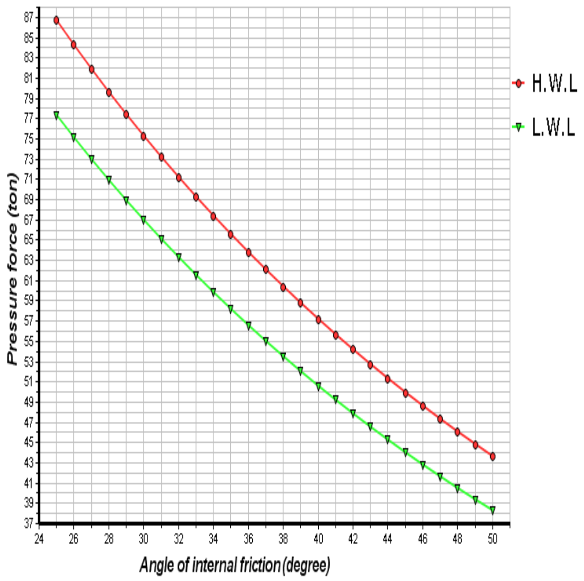


Fig. 2. Relationship between lateral pressure force acting on the bottom block and the angle of internal friction (seismic case).

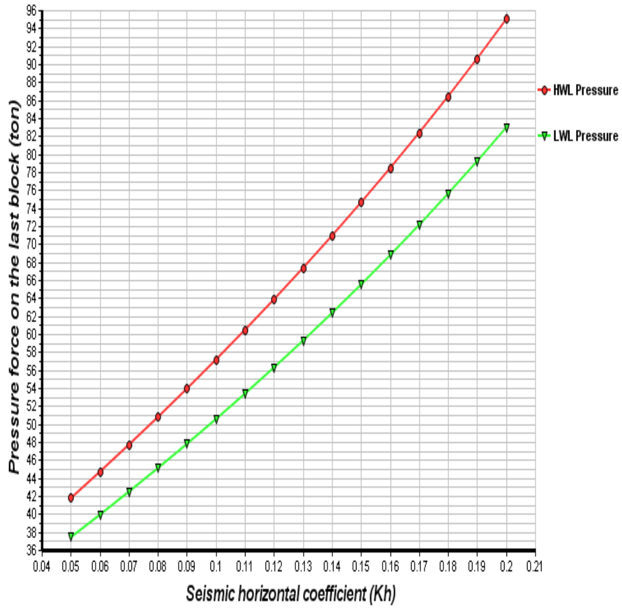


Fig. 3. Effect of seismic horizontal coefficient force.

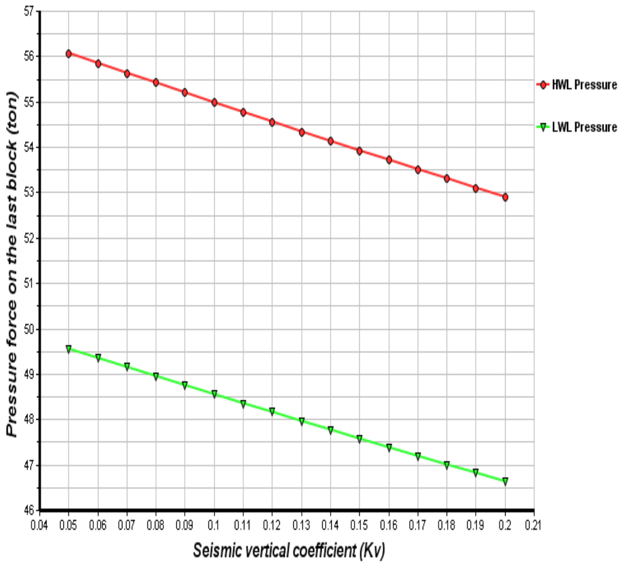


Fig. 4. Effect of seismic vertical coefficient force.

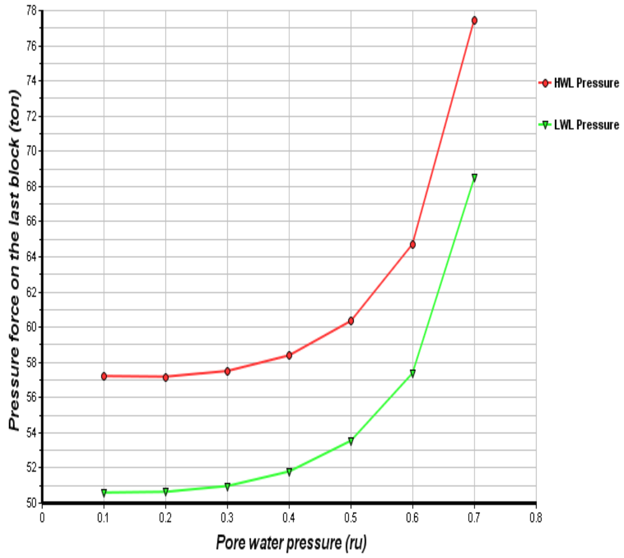


Fig. 5. Effect of pore water pressure on lateral pressure.

Table 2
Parameters implemented in design charts of groups (1), (2) and (3), (static case)

| Parameter | Chosen value | | |
|---|---|---|---|
| Group number | Group(1) | Group(2) | Group(3) |
| Case | Static | Static | Static |
| Draft (m) | 4 – 9 | 9.5 – 15 | 15.5 – 20 |
| Clearance (m) | 1 | 1 | 1 |
| Tidal range (m) | 0.5 | 0.5 | 0.5 |
| Distance between upper surface of head and H.W.L (m) | 1.5 | 1.5 | 1.5 |
| Lower surface level of the head (m) | +0.5 | +0.5 | +0.5 |
| Height of each block (m) | 1 .75 | 2.25 | 2 .75 |
| Saturated unit weight of concrete (ton/m$^3$) | 2.2 | 2.2 | 2.2 |
| Buoyant unit weight of concrete (ton/m$^3$) | 1.2 | 1.2 | 1.2 |
| Friction coefficient (concrete/ concrete) | 0.4 | 0.4 | 0.4 |
| Friction coefficient (concrete/ rock) | 0.5 | 0.5 | 0.5 |
| Angle of internal friction of soil (degree) | 30 | 30, 45 | 45 |
| Angle of internal friction of back fill (degree) | 45 | 45, 45 | 45 |
| Average dry unit weight of soil (ton/m$^3$) | 1.85 | 1.85 | 1.85 |
| Average buoyant unit weight of soil (ton/m$^3$) | 0.9 | 0.9 | 0.9 |
| Bearing capacity of soil (ton/m$^2$) | 12.5, 15, 17.5 | 20, 25, 30 | 30, 35 |
| Buoyant unit weight of rock (ton/m$^2$) | 1 | 1 | 1 |
| Angle of interface friction (soil/structure) (degree) | 0 | 0 | 0 |
| Pulling force on bollard (ton/m) | 2 | 2 | 2 |
| Live load (ton/m$^2$) | 2 | 2 | 2 |

## 5. Conclusions

A simplified method has been introduced to account for dynamic earthquake forces in the analysis of plain concrete blocks quay wall. The analysis includes all factors and provides complete set of data and a complete drawn design charts. From analysis and the charts the following may be concluded:

1. Lateral force may be increased by a percentage ranges from 10% to 20% due to seismic effect.

2. Vertical seismic coefficient can be disregarded and both horizontal seismic coefficient and pore pressure can be taken as 0.1 in our region.
3. A complete dynamic analysis including damping and inertia should be carried out to give more rigorous behavior of berth under different dynamic loads.
4. Dynamic factors should be accounted for, even for zones of little hazard risks.
5. It is clear that the second block from the bottom is usually the longer block particularly in deep berths analysis and semi strong soil.
6. Ship draft, backfill characteristics and strength of bearing capacity stratum are the most important factors affecting design of plain concrete blocks quay wall.

Table 3
Parameters implemented in design charts of groups (4), (5) and (6), (seismic case)

| Parameter | Chosen Value | | |
|---|---|---|---|
| Group number | Group(4) | Group(5) | Group(6) |
| Case | Seismic | Seismic | Seismic |
| Draft (m) | 4 – 9 | 9.5 – 15 | 15.5 – 20 |
| Clearance (m) | 1 | 1 | 1 |
| Tidal range (m) | 0.5 | 0.5 | 0.5 |
| Distance between upper surface of head and H.W.L (m) | 1.5 | 1.5 | 1.5 |
| Lower surface level of the head (m) | +0.5 | +0.5 | +0.5 |
| Height of each block (m) | 1 .75 | 2.25 | 2.75 |
| Horizontal seismic coefficient as a friction of g (kh) | 0.1 | 0.1 | 0.1 |
| Vertical seismic coefficient as a friction of g (kv) | 0 | 0 | 0 |
| Excess pore water pressure ratio (ru) | 0.1 | 0.1 | 0.1 |
| Saturated unit weight of concrete (ton/m$^3$) | 2.2 | 2.2 | 2.2 |
| Buoyant unit weight of concrete (ton/m$^3$) | 1.2 | 1.2 | 1.2 |
| Friction coefficient (concrete/ concrete) | 0.4 | 0.4 | 0.4 |
| Friction coefficient (concrete/ rock) | 0.5 | 0.5 | 0.5 |
| Angle of internal friction of soil (degree) | 45 | 45 | 45 |
| Angle of internal friction of back fill (degree) | 45 | 45 | 45 |
| Average dry unit weight of soil (ton/m$^3$) | 1.85 | 1.85 | 1.85 |
| Average buoyant unit weight of soil (ton/m$^3$) | 0.9 | 0.9 | 0.9 |
| Bearing capacity of soil (ton/m$^2$) | 12.5, 15, 20 | 25, 30 | 35 |
| Buoyant unit weight of rock (ton/m$^2$) | 1 | 1 | 1 |
| Angle of interface friction (soil/structure) (degree) | 0 | 0 | 0 |
| Pulling force on bollard (ton/m) | 2 | 2 | 2 |
| Live load (ton/m$^2$) | 2 | 2 | 2 |

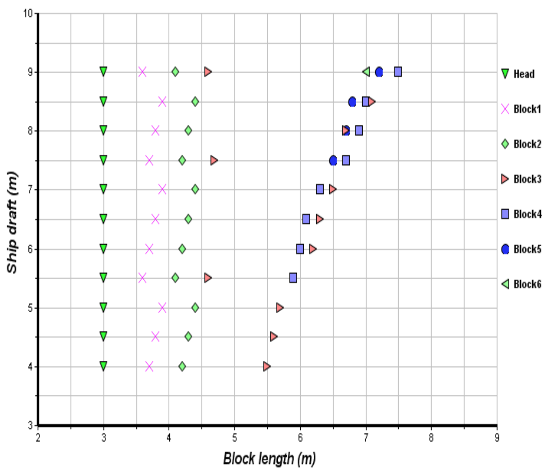


Fig. 6. Design charts group (1). bearing capacity = 12.5 (ton/m$^2$). (static case).

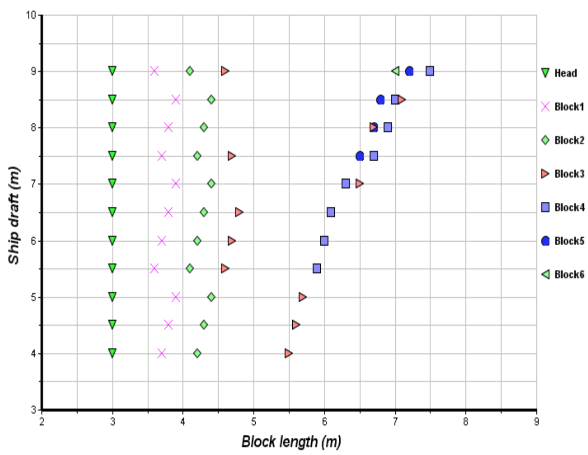


Fig. 7. Design charts group (1). bearing capacity = 15 (ton/m$^2$). (static case).

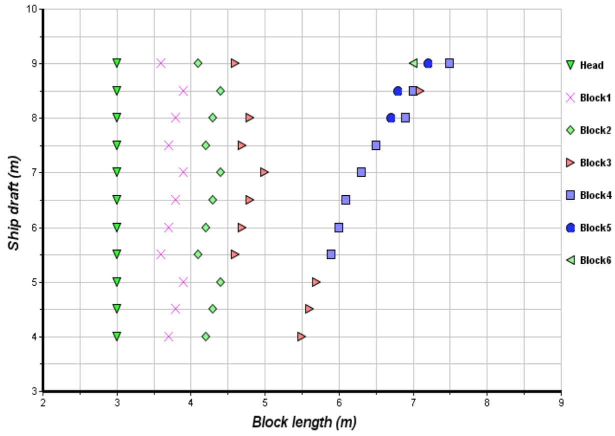


Fig. 8. Design charts group (1) bearing capacity = 17.5 (ton/m²). (static case).

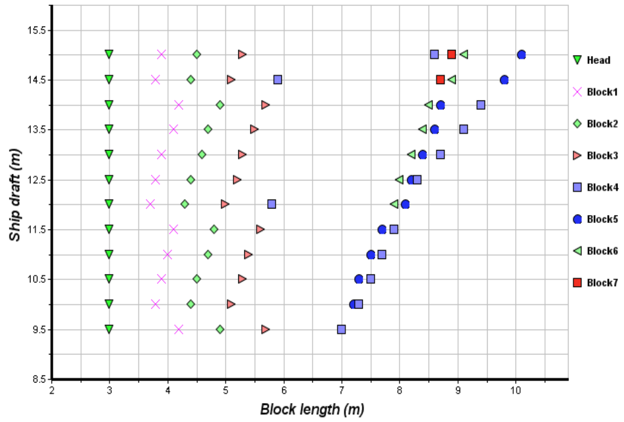


Fig. 11. Design charts group (2) bearing capacity = 30(ton/m²), angle of internal friction of backfill = 40° (static case).

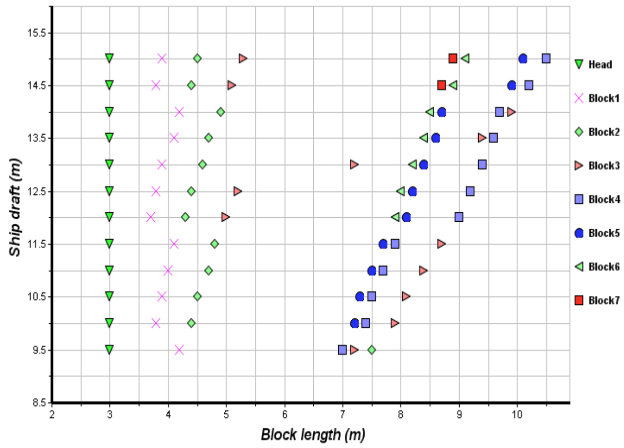


Fig. 9. Design charts group (2) bearing capacity = 20 (ton/m²), angle of internal friction of backfill = 40° (static case).

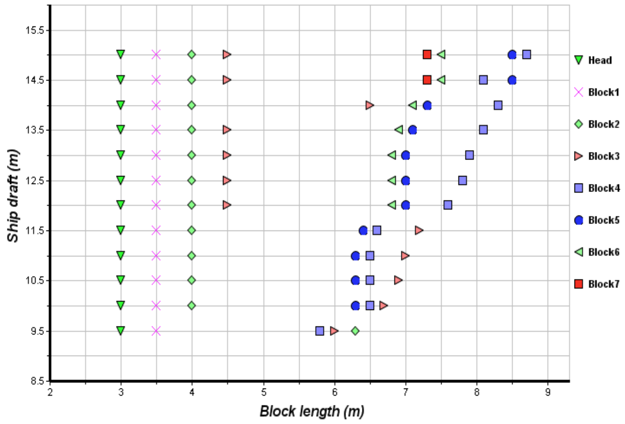


Fig. 12. Design charts group (2) bearing capacity = 20 (ton/m²), angle of internal friction of backfill = 45° (static case).

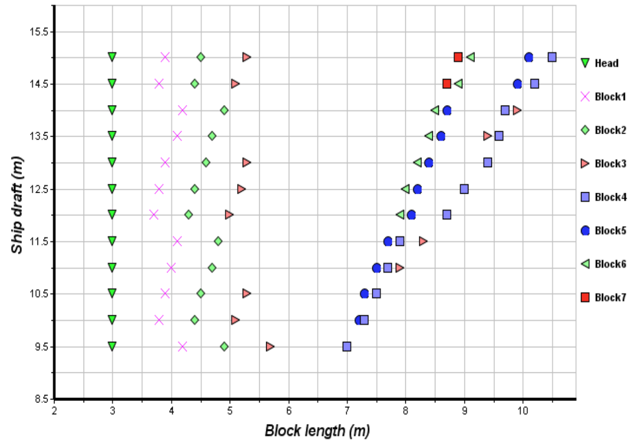


Fig. 10. Design charts group (2) bearing capacity = 25 (ton/m²), angle of internal friction of backfill = 40° (static case).

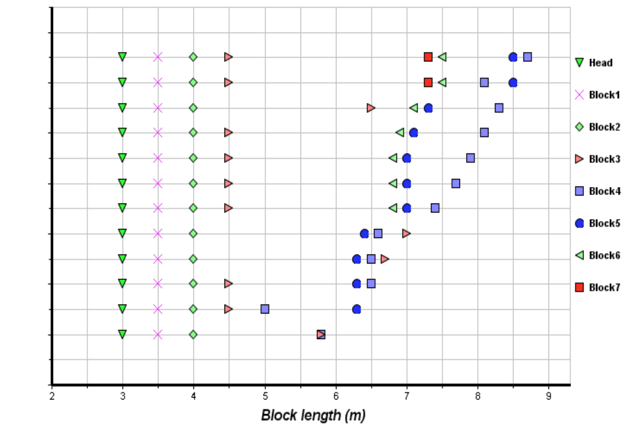


Fig. 13. Design charts group (2) bearing capacity = 25 (ton/m²), angle of internal friction of backfill = 45° (static case).

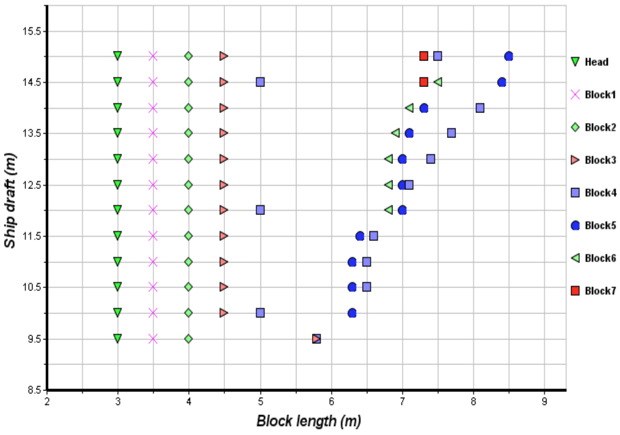


Fig. 14. Design charts group (2) bearing capacity = 30 (ton/m$^2$), angle of internal friction of backfill = 45$^o$ (static case).

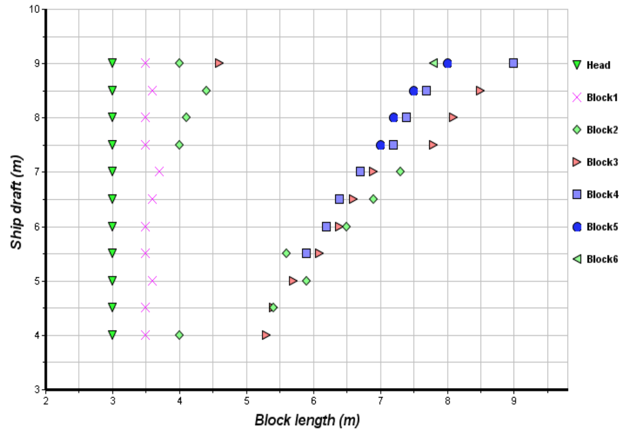


Fig.17. Design charts group (4) bearing capacity = 12.5 (ton/m$^2$). (seismic case).

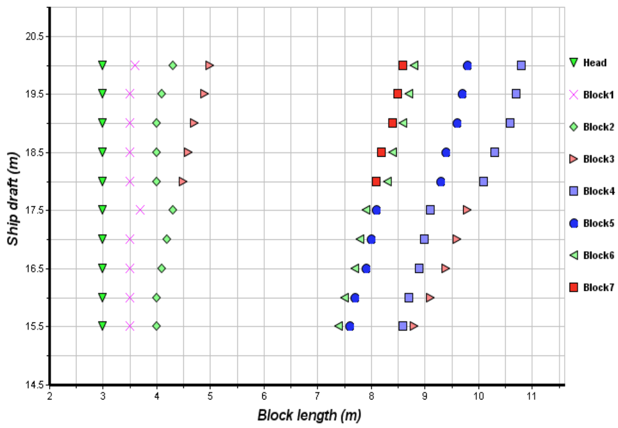


Fig. 15. Design charts group (3) bearing capacity = 30 (ton/m$^2$), maximum tidal range = 0.5(m) (static case).

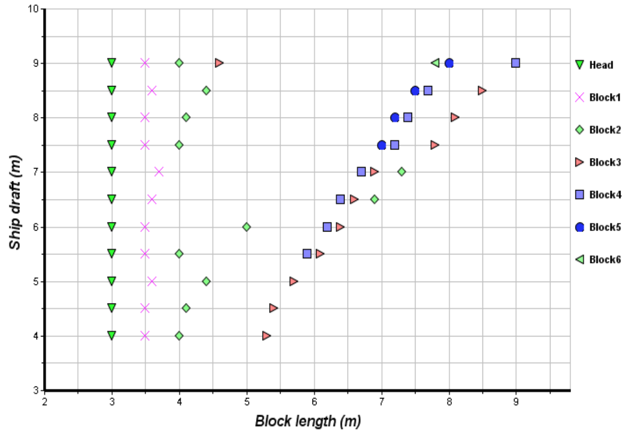


Fig. 18. Design charts group (4) bearing capacity = 15 (ton/m$^2$) (seismic case).

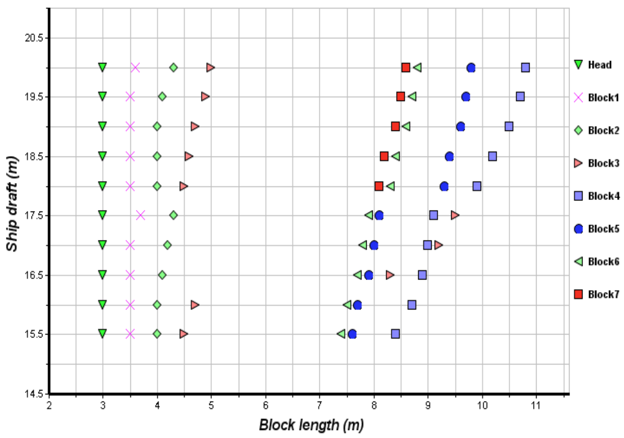


Fig. 16. Design charts group (3) bearing capacity = 35 (ton/m$^2$), maximum tidal range = 0.5(m) (static case).

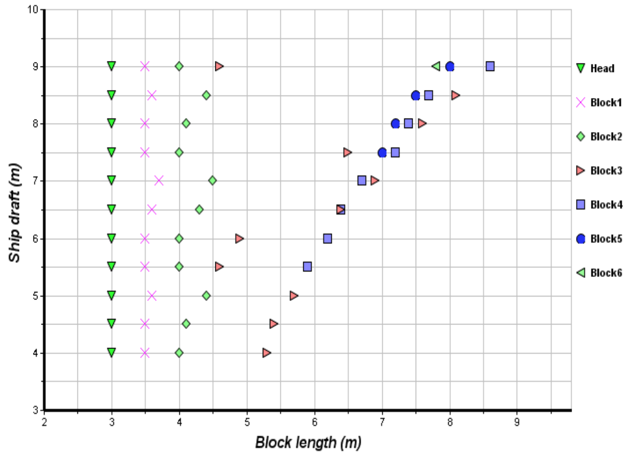


Fig. 19. Design charts group (4) bearing capacity = 20 (ton/m$^2$) (seismic case).

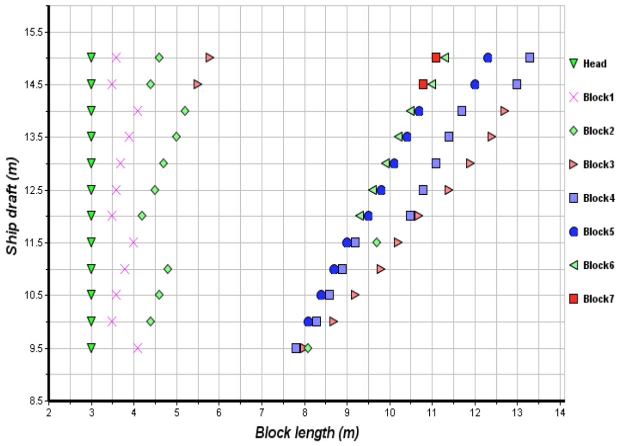


Fig. 20. Design charts group (5) bearing capacity = 25(ton/m²), maximum tidal range = 0.5(m) (seismic case).

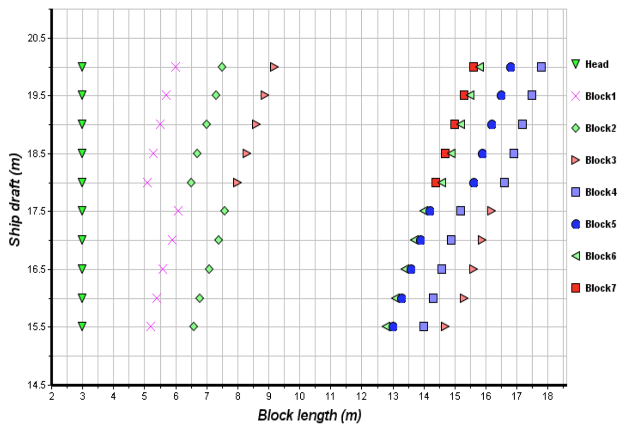


Fig. 21. Design charts group (5) bearing capacity = 30 (ton/m²), maximum tidal range = 0.5(m) (static case).

7. Design charts are applicable for concrete blocks quay walls within the range of parameters specified. For other cases and parameters, more analysis should be calculated.
8. In case of tidal range other than that specified, the height of the quay wall may be adjusted and charts still can be used.

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