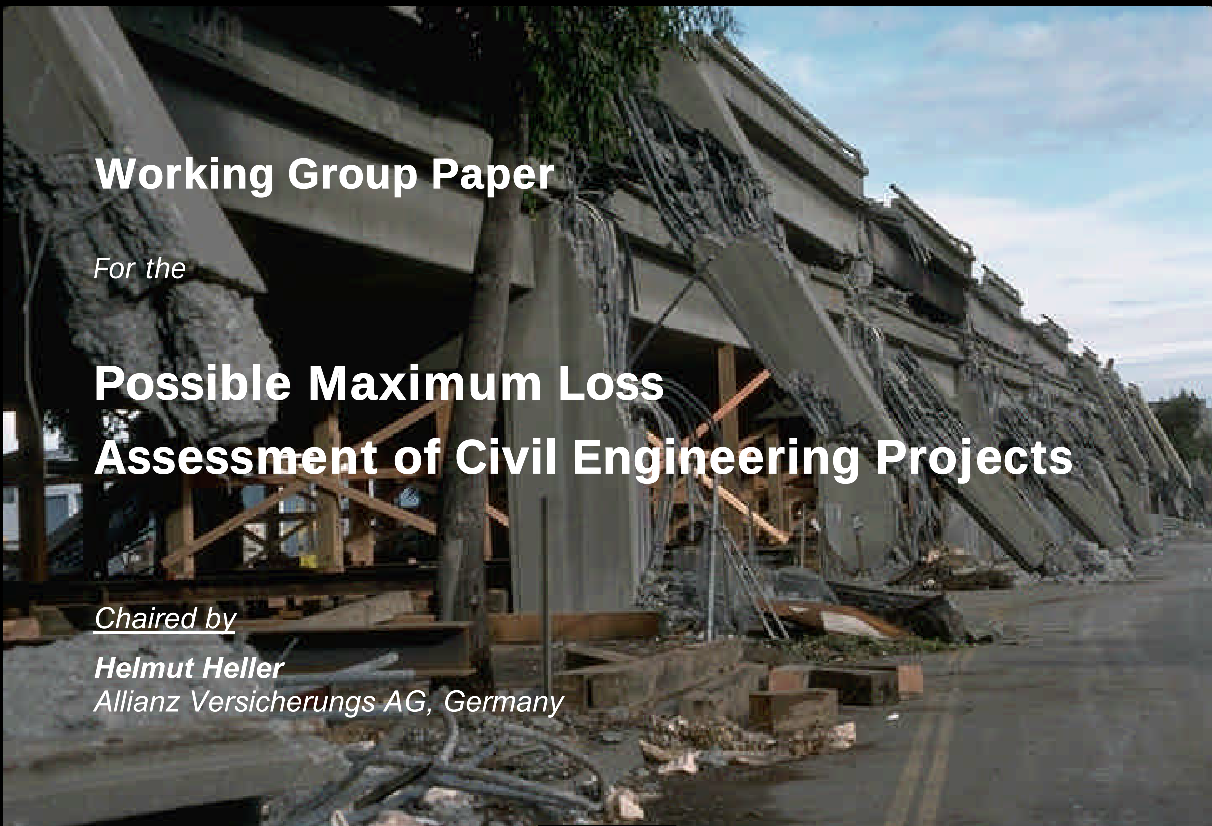

Working Group Paper

For the

Possible Maximum Loss Assessment of Civil Engineering Projects

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1 Introduction & Summary

The last IMIA publication concerning PMLs was presented at the Madrid Conference in 1993, entitled "*New Aspects of PML Estimation in CAR & EAR Insurance*". The last ten years of construction insurance has been plagued by catastrophic losses, the most noteworthy for tunnelling, that demand a new appraisal of this topic. The 1993 document mainly considered the generic hazards at stake, common to all construction projects, and the principal factors governing these.

This is the first of any such paper that considers the sensitivity of each type of technical risk for the assumed most unfavourable hazards from which the maximum damage can be determined. The typical components of the principal types of civil engineering construction risk are introduced to highlight their main characteristics and differences in sensitivity for the hazards discussed. Suggested PML scenarios are also given for guidance, reflecting in some cases historical losses.

This paper does not deal with operational risks, contractors' plant & equipment, third party risks, EAR risks (even if part of an engineering project), Loss of Profits, or indeed the potential magnitude of any PML.

2 Definitions

The question of finding a uniform definition and the necessity to determine the main parameters for establishing a PML has been treated in numerous studies and publications and discussed in many committees and panels. As a result, definitive guidelines for determining the PML have been introduced in markets and insurance companies. Nonetheless, a variety of abbreviations are used in international markets and it is not uncommon for an insurer / reinsurer writing an international portfolio to be presented with the following expressions:

PML	*Probable Maximum Loss*
PML	*Possible Maximum Loss*
MPL	*Maximum Possible Loss*
MPL	*Maximum Probable Loss*
EML	*Estimated Maximum Loss*
MFL	*Maximum Foreseeable Loss*
CML	*Credible Maximum Loss*
MAS	*Maximum Amount Subject etc.*

In reality many of these expressions are similar in that they establish a maximum loss amount. However, the problem with interpreting a definition is the first element within the term, i.e. the word estimated, foreseeable, probable, or possible. Particularly the letter "P" in PML / MPL with the alternative meaning of "probable" and "possible" exacerbates the scope of confusion. Thus both terms need to be considered further to illustrate the differences.

2.1 Probable Maximum Loss (PML)

In 1975 a Working party of the International Machinery Insurers Association (IMIA) made an attempt to find the most suitable guidelines for PML evaluations. The group describes the PML as follows:

"The PML is an estimate of the maximum loss which could be sustained by the insurers as a result of any one occurrence considered by the underwriter to be within the realms of probability. This ignores such coincidence and catastrophes as may be possibilities, but which remain highly improbable".

The “9/11” World Trade Centre disaster highlighted just how vulnerable a “probable” rationale may become, since such events were not even within peoples’ scope of imagination. Thus some insurers have given much consideration to this philosophy, and reverted back to adopting a more restrictive “possible” only rationale.

2.2 Possible Maximum Loss (PML/MPL)

The following definition is widely used by property & fire insurers:

“*The Possible Maximum Loss is the largest loss that may be expected from a single fire (or other peril when another peril may be the controlling factor) equal to any given risk when the most unfavourable circumstances are more or less exceptionally combined and when, as a consequence, the fire is unsatisfactorily fought against and therefore is only stopped by impassable obstacles or lack of sustenance.”*

In reality one has to be aware that there may be a fundamental difference between the two definitions. Any rationale must consider all conceivable negative and thereby even improbable circumstances that may accumulate in a particularly unfortunate way. In the case of an individual risk the Possible Maximum Loss will thus equal 100% of the sum insured.

It is naturally at the discretion of the individual underwriter to decide which definition best reflects the philosophy of his company. However, whatever term is used, it is imperative that any calculation be accompanied by definitions of all terms used, a clear statement of the rationale applied and assumptions made.

It is also essential of course to determine the PML according to sound underwriting data and, wherever possible, by involving expert engineers particularly for large industrial risks.

3 Assessment Process

A basic process must be followed in order to assess and secure the appropriate PML for a particular risk. The process detailed following flowchart offers guidance on each stage as considered in the various sections of this paper.

What is being built and how?	Assemble sufficient information to be able to establish the construction methods and processes to be employed, and what is actually being built. (***see Section 4***)
What do the works cost to build?	Ensure that there is sufficient cost data available, with a breakdown of cost for various elements of the project. (***see Section 5***)
What could cause damage to the works?	Assess the hazards / perils that could cause physical damage to the works e.g., earthquake, fire. (***see Section 6***)
What is the Maximum Loss scenario?	What is the most catastrophically possible event within the bounds of reason, which could give rise to the maximum physical loss or damage to the works? (***see Section 6***)
What is the value (PML) associated with the loss scenario?	Taking all things into consideration what is the cost of reinstating the portion of works lost or damaged in the PML scenario? (see Section 7)

N.B. *This may be considerably more than the original cost of the damaged element of the works. Where this is the case advice should be sought from the underwriter as there may be policy limitations.*

4 Information Required To Undertake a PML Assessment

In order to be able to assess a project for a PML calculation, the following information, as a minimum, is needed:

1. An **overall plan of the project** indicating its position relative to external physical attributes such as adjacent property, topographical features and environmental features.

   This information will be used to assess natural peril hazards and those arising from the concentration of values on the project.

2. Plans and sections of key structures on the project. These should identify construction materials, structural layout, construction sequences (with complex structures) and temporary works requirements.

For projects with significant temporary works, the basic design criteria should be supplied to allow the Risk Engineer to assess the risk exposure in the temporary state e.g., design criteria for marine cofferdams and if allowance has been made for suitable flood return periods.

3. Construction cost details for all major components of the project, where possible broken down to an elemental or component level or trade contractor level. For further information see Section 5.

4. Construction programme for the project. This can be used to assess the exposure of the project to seasonal perils (e.g., winter floods) and should include start & finish dates, in particular to highlight the duration of high-risk activities.

5. Schedule of key plant, the associated values and where Advanced Loss of Profits (ALOP) is included an estimate of the time to replace any item.

5 COST DATA

The original cost data will be used to establish the reinstatement cost of the damaged section of the works following the occurrence of a PML event. In addition there may be additional construction processes required to reinstate the works e.g., ground improvement following a tunnel collapse.

The data required from the client to permit calculation of a PML should include, as a minimum:

- *Total Project Cost*
- *Breakdown of cost for major elements*
- *Value of any free-issue materials*

All PMLs should be calculated in the original contract works or policy currency and converted to the applicable treaty, or facultative reinsurance currency. A note should be made of the exchange rate at the time of calculation and this should be highlighted to the underwriter. If a currency appears volatile then a recommendation may be appropriate to allow for the currency risk.

5.1 References

If there are external sources of cost data available, these may be useful when considering the cost of reinstating certain elements of a scheme e.g., ground freezing to rebuild a collapsed section of a tunnel.

Examples of cost reference materials include:

- *Civil Engineering & Highways Pricing Book (Compiled by Davis Langdon and Everest – Firm of QS Consultants and published by “SPONS”) - a database of element cost data*
- *The Royal Institute of Chartered Surveyors (RICS) web page (http://www.rics.org.uk/) that has some limited cost data*
- *Trade press that has articles on the current project.*

6 PML CONSIDERATIONS

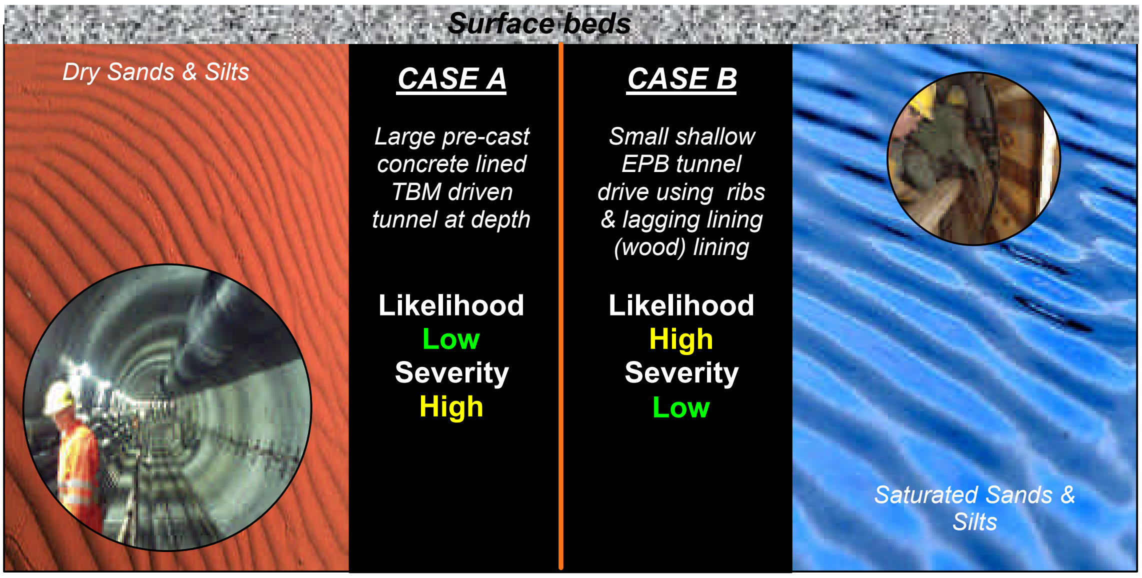
6.1 Tunnelling

6.1.1 Introduction

Whilst every segment of construction has to contend with particular hazards and associated risks, there is none which compares to the range and exposures plaguing the tunnelling industry. This section deals with underground construction involving the creation of a permanent underground void, or passage, using safe excavation procedures to ensure the stability of the ground during construction and after.

Whilst the PML considerations in this section will be dealt with according to the methods of excavation and type of lining support, it must be recognised that the criteria governing the adopted tunnelling method will also influence the likelihood and severity of the potential hazards. These additional factors relate to size, purpose, depth, programme & cost constraints, prevailing geological / hydrological conditions and the impact on (or from) existing structures or installations either at surface or underground. Thus likelihood and severity considerations are complex, and care is required in selecting a sound PML rationale.

An example of the relationship between the two for a tunnelling project involving one large and one small tunnel is given below, with an assumed lining failure PML scenario.



Thus the “likelihood” and “severity” of the two examples above depend on the size of the tunnel, the lining used, its depth, and the properties of soil or ground medium through which the tunnel is to be driven. The PML will be higher in *Case A* as opposed to *Case B* even though the likelihood will be much less. The two cases could have been demonstrated with almost infinite combinations of ground conditions & tunnelling methods. It is thus beyond the scope of this document to discuss all the criteria that may affect a PML consideration, and it is essential to seek expert opinion to help identify the correct scenario that best reflects the chosen PML definition.

6.1.2 Construction / Excavation Methods

A major infrastructure project will often involve a number of components, possibly including tunnel construction either as a small component of the overall scheme, or many tunnel components effected with varying methods of excavation and lining support.

6.1.3 Classification

Tunnels can be classified according to purpose (transportation, conveyance or storage) once completed, or construction-wise according to the type of ground involved (soft ground, hard rock, submersed tube), the primary method of excavation (Tunnel Boring Machine - TBM, drill and blast) or indeed the method of support (New Austrian Tunnelling Method - NATM, segmental linings, shotcrete tunnels). The generally accepted generic descriptions of the most common tunnelling methods are:

- *New Austrian Tunnelling Method - NATM*
- *Tunnel Boring Machine Method – TBM (Shields & Tunnel Boring Machines including Pipejacking method)*
- *Hand driven tunnels*
- *Drill & Blast*
- *Submersed Tube*

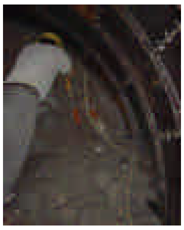
Whatever classification is chosen to describe the “type” of tunnel, there will always be an indicative method of safe excavation to be taken into consideration according to the type of permanent lining or temporary support employed. The classification used in this paper only distinguishes between the methods of excavation and tunnel linings / methods of support, being chosen to highlight the severity of hazard exposures peculiar to each, as opposed to the more common terms given above. Any method that cannot be identified according to the classifications given in 6.1.4 & 6.1.5 should be treated as prototype technology, whereby the PML should assume an MFL value of 100% of the indemnity provided.

This section does not consider the numerous types of shafts, their linings, sinking methods or related hazards. Shafts alone will not form the primary basis of a major tunnelling project PML, but may have a significant impact on the final figure and should therefore be taken into consideration with the scenarios presented in section 6.1.8.

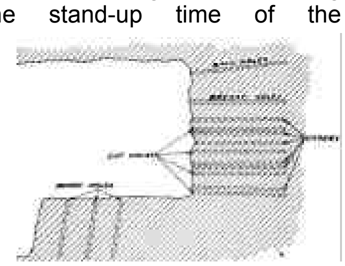
6.1.4 Indicative methods of Excavation

1. *Conventional*
Mechanical Excavation with roadheader cutting boom, or backhoe shovel (Including NATM). Many roadheaders will include flight conveyor loading equipment for continuous spoil removal from the face, rubber tyre front end loaders are alternatively employed.

2. *Manual excavation* (hand mining) for timber lined headings or with steel / cast iron liners. Loading performed by hand into rail mounted skip cars pushed out to the shaft.



3. *Drill and Blast* The technique is exclusively used in rock tunnels where the ground conditions are sufficient for the already completed sections to resist against the blasting vibrations. The stand-up time of the unsupported span must also enable installation of any support systems that may be required - or otherwise in accordance with the NATM philosophy (described below).



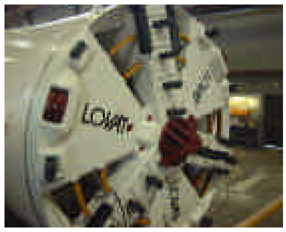
4. *Open cut excavation,* spoil removed directly to surface by excavator or otherwise mechanical loader. Open cut excavations often involve a variety of other civil engineering techniques not detailed in this section.

5. *Open Face Shield* A protective "shield" is basically open ended steel cylinder long enough to be equipped with provisions for face support (hydraulic breasting and or poling plates) and propulsion cylinders, generally dimensioned slightly longer than the unit length of tunnel lining. Excavation is either by hand, or with a roadheader, "backactor" or backhoe.

6. *Closed Face mechanical excavation shield; Air Pressurised Shield –* very rare, as above but with an airlock and bulkhead to permit the "excavation chamber" to be pressurised with compressed air to aid face support.

7. *Slurry machine* These machines separate the face from the rest of the shield and machine behind by a pressurised bulk head. The spoil excavated by the cutting wheel at the face is removed by the parallel charging and discharge of benonite slurry to and from the face.

8. *Earth pressure balance machines (EPBM)* These machines are a further development of slurry machines, similar but basically without the slurry. Instead of providing face support with a benonite medium, the spoil itself is maintained under pressure in the cutting head and removed in "closed mode" by a screw conveyor to maintain pressure at the face. The key feature of the method is the screw conveyor by which the transported material is further compressed to form an impervious plug of spoil to prevent the free passage or escape of water and spoil along its length.



9. *Full Face Hard rock Tunnel Boring Machine – NO shield*. Hard Rock TBMs are often not fitted with a full shield or pressurised bulk head to protect against subsidence or face collapse where good stable ground conditions are expected.
These machines are vulnerable to unforeseen geological surprises (jointing, fissures, faults) leading to unstable face conditions or water ingress, and emergency action may be difficult because of the restricted forward access to the face. Machines without any provision for probing head, or otherwise grouting ahead of the face, are even more vulnerable and remedial action can only be taken from the surface.

10. *Hybrid Hard Rock / Soft Ground TBM - Full Face with shield.* Hybrid machines employ a cutting head and loading facilities cater for both soft rock (or soft ground) as well as hard rock conditions. They are designed to negotiate heterogeneous beds with mixed face conditions (danger of settlement!)

11. *Remote control microtunnelling* technology. Not detailed.

12. *Submersed tube* with or without prior dredging. Not detailed.

6.1.5 Tunnel support and lining systems

1. *Pre cast segmental concrete linings* These linings are usually employed with TBM driven tunnels sized slightly smaller (˜ 5cm) than the excavated diameter, and grouted in place once erected behind the TBM or shield.

2. Jacked concrete pipes Used exclusively with TBM or shield drives. Instead of being installed at the TBM similar to segmental linings, they are installed at the start shaft and jacked into the tunnel to "follow" the TBM. Thus during the jacking process the whole tunnel moves forward as the TBM progresses.

3. *Rolled Steel Ribs and Lattice Arches (used with Drill & Blast / NATM)* There are two basic forms of steel arch, depending on size, required geometry, and purpose. Rolled steel arches can be used on their own for immediate and or permanent support, usually combined with steel lagging, rock bolts etc., whilst lattice aches are, once installed, embedded within sprayed concreted or shotcrete linings.

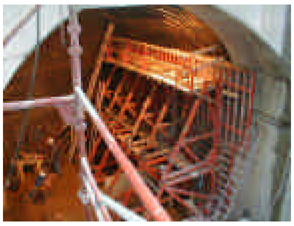
4. *Rock bolts (used with Drill & Blast / NATM)* Used to retain the excavated, or mined, surface to prevent movement across, or failure due to natural discontinuities such as faults bedding planes and fissures.

5. *NATM* The **N**ew **A**ustrian **T**unnelling **M**ethod is not a type of lining but a specialised technique to provide flexible support to an excavation. The primary lining support (shotcrete / steel arches / wire mesh) is allowed to deform with the excavated void so that the loads are transferred to the lining until a state of structural equilibrium is attained. The excavation design should be tailored to the limitations of the ground and method of support. This means in soft ground for example that large excavations must be excavated in

separate drives each reaching a state of equilibrium before the next is begun. This allows the ground loads to be progressively redistributed, into and around the lining, with each "enlargement" of the tunnel profile. Monitoring of deformations is paramount to the success of the method, to so permit the timely application of any remedial action that may become necessary. A secondary (permanent) lining should only be installed once the deformation process has ceased.

It should also be noted that NATM has little application in very poor ground with no or with a very short stand-up time. Whilst a shotcrete lining can (and often has been) be used in such ground conditions the design of such applications is not in line with the true NATM technique (Prototype?)

6. In situ concrete linings (also NATM lining) In-situ concrete linings describe both the shuttered and sprayed (shotcrete) methods of installation. Whilst shuttered concrete is poured behind formwork, sprayed concrete or shotcrete is applied pneumatically using compressed air to transport a wet or dry mix through a flexible hose to the application nozzle.



7. *Compressed Air* Whilst lining components such as steel ribs and shotcrete provide structural support to the inside of the tunnel, compressed air is used to "support" the tunnel face, in particular to combat water ingress.

8. *Ribs & lagging wooden lining with steel rings.* This method is used exclusively as a temporary (primary) lining in soft ground tunnelling with a shield or TBM, in particular in the USA and Canada. Wooden boards are placed behind circular steel ribs which are expanded against a geotextile membrane (porous) and the excavated surface behind. A secondary concrete lining is installed in a separate operation on completion of the drive.

6.1.6 Excavation Sensitivity Factors

Excavation Method	**Sensitivity Factors ***					
	EQ	Flood (external)	Flood (at face)	Fire	Explosion	Face collapse, inadequacies of method for unforeseen conditions
1 Conventional	3	2	3	2	2	3
2 Manual	3	1	3	2	2	3
3 Drill & Blast	3	1	3	2	1	3
4 Open Cut	2	2	2	1	1	2
5 Open face Shield	3	2	3	2	2	3
6 Closed Face Shield	3	2	2	2	2	2
7 Slurry TBM	3	2	1	2	3	2
8 EPBM	3	2	1	2	2	2
9 Full Face Hard Rock TBM – no shield	3	2	3	2	2	3
10 Hybrid TBM	3	2	3	2	2	2
11 Micro Tunnelling	3	2	3	3	3	3
12 Submersed Tube	3	3	-	2	2	3

Severity Factors
0 – Excavation Method unaffected
1 – Minor influence on method – drive method can be maintained once hazard relieved
2 – Major Failure of face & or drive – may require alternative working method
3 – Catastrophic failure of face, possible abandonment of drive.

6.1.7 Lining Sensitivity Factors

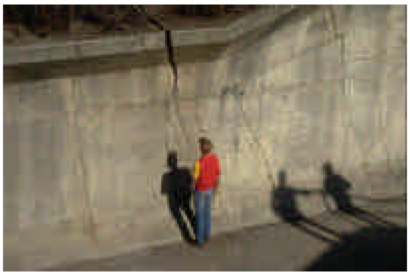
Tunnel Support & Lining Systems (fully installed)	**Sensitivity Factors ***				
	EQ	Flood	Fire	Explosion	Inadequate design or method of execution: lining failure / collapse
1 Pre-cast Segmental Linings	2	2	1	2	3
2 Jacked Concrete Pipes	2	2	1	2	3
3 Steel Ribs & Lattice Arches (NATM)	2	1	1	2	3
4 Rock Bolts (NATM)	2	0	0	0	3
5 Shotcrete also NATM	3	1	2	2	3
6 In-situ concrete Linings (inc. NATM)	3	1	1	2	3
7 Compressed Air (Face Support)*	2	2	3	3	3
8 Ribs & Lagging	2	2	3	3	3
7 Contiguous Piles (Open Cut)	2	0	0	0	2

Severity Factors
0 – Lining or support method unlikely to suffer damage
1 – Minor damage (or localised) to lining or support systems– can be repaired
2 – Significant damage to support systems or lining – may require alternative working method for repair
3 – Catastrophic failure of tunnel, possible abandonment
** - Compressed Air only used for Face Support, in combination with other lining methods.*

6.1.8 PML Scenarios

The PML scenarios that can be foreseen for tunnelling projects are in theory numerous. The exceptional scenarios generated on a few projects in particular suggests that only a few are indeed appropriate for reinsurance purposes. The Heathrow, Hull, Great Belt and Anatolian Motorway projects revealed completely new loss circumstances that were beforehand considered very unlikely if not unreasonable. If underwriters are to surrender to the lessons learned on these projects then the only reliable PML for soft ground tunnelling is going to reflect the magnitude (?60-160Mn) of the material damage claims experienced on these projects, or otherwise equal to any sub-limits applied for underground construction. Most projects will have enough in common with the scenarios described below for these to be taken as a tenable basis, irrespective of specific design details or the exact nature of the ground conditions prevailing on the project alignment.

6.1.8.1 Earthquake



Tunnels have been “traditionally” considered less sensitive to the effects of earthquake than surface structures, but no tunnel can resist a bisecting fault, or near fault effects, or indeed any which has not been designed for. The recent (1999) earthquake damage suffered by a tunnel on the Anatolian Motorway project, led to a hitherto unknown scenario resulting in a partial abandonment and re-routing of the drive.

The dynamic response of underground structures in soils with variable mechanical properties can seldom be predicted with confidence, a situation made worse by the fact that very few tunnels include a full earthquake design analysis. *A design that actually caters for an earthquake event, is not particularly relevant for a PML calculation since it will only consider the completed structure, and not one which is only partially installed during the drive.*

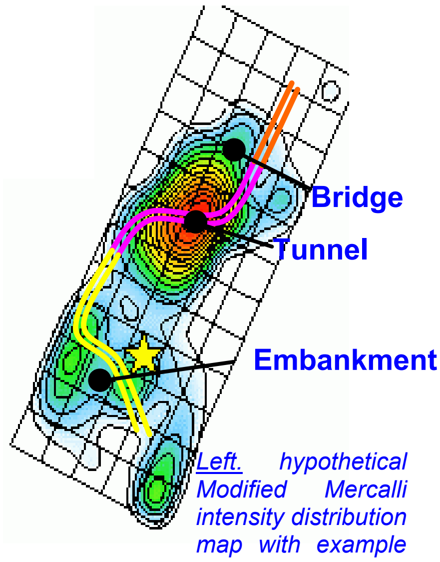
The further difficulty in considering the impact of earthquakes on tunnels regards the scale of the scheme under consideration. A large project will often involve many project components, all of which can be affected by one earthquake event. It would therefore not be valid to apply an earthquake scenario to the most sensitive component, since a PML must take all the components into consideration, besides just tunnels. Indeed scenarios other than earthquake may prove more applicable in some cases, but the impact of earthquake on the whole scheme must always be checked.

Stand Alone Tunnels This scenario considers one tunnel (portal to portal / shaft to shaft). NATM tunnels with multiple headings must take all the tunnel faces and headings into consideration, the worst case always during the construction period with the highest number of parallel or simultaneous intermediate stages in progress.

Tunnel Face: The tunnel face will either remain unaffected or suffer catastrophic failure leading to uncontrollable loss of ground and ingress of water. The drive may have to either be partially abandoned back to a suitable juncture or in full. A suitable juncture may be considered where the tunnel is in dry stable ground conditions, or at an alternative node or point of access such as an intermediate shaft, cross passage etc.

Tunnel Lining: If several types of tunnel lining are foreseen then each must be considered separately. The worst case will always consider damage to the longest approximately straight leg (i.e. the longest section in any one particular direction ±20º) assuming the worst orientation and response to “inbound” shock waves.

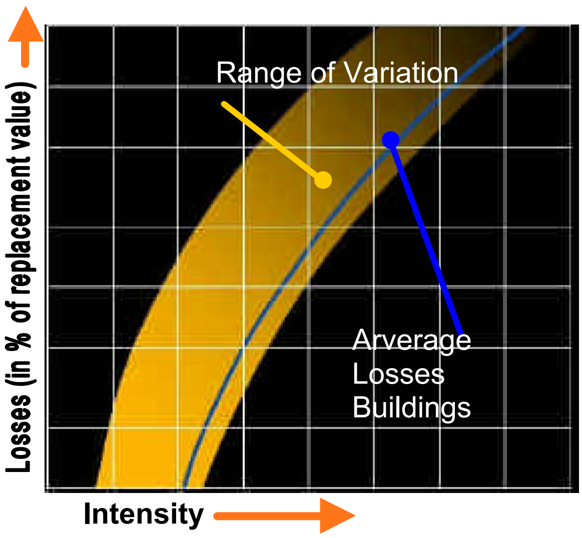
Separate subsidiary faults should also be assumed to develop across the tunnel from known faults within 10-15km of the alignment. The effect of a direct hit should assume a displacement sufficient to necessitate realignment of the drive or abandonment and re-routing, depending on ground conditions. Additional flood damage may occur due to lining failure.



Left. hypothetical Modified Mercalli intensity distribution map with example project alignment.

Major projects: The most suitable earthquake scenario for a major infrastructure project is demonstrated below. The hypothetical project comprises a series of tunnels, viaducts and embankment components.

The PML can be evaluated by taking each component separately, and estimating the damage according to a hypothetical Modified Mercalli intensity distribution of an earthquake. An overall assessment can be made from the Munich Re percentage loss against EQ intensity graph. The scenario must however additionally account for specific localised damage due to "direct hits" from faults leading to lining failures or collapse as highlighted above.



Earthquake Damage Estimation Range		
Earthquake Intensity	%age Damage	
	Maximum	Minimum
10	100	33
9	100	15
8	53	8
7	27	5
6	11	3,5
5	5	3
4	6	3
3	0	0
2	0	0
1	0	0

6.1.8.2 Flood

If the project site is exposed to a flooding hazard, or if the tunnel is to be driven below a water table with little protection by stable impermeable layers above the crown, then catastrophic water ingress is to be considered.

The possible scenarios are, depending on the excavation, support, and or lining methods:


Flooded Great Belt tunnels TBM

a) water damage to the underground equipment and installations
b) a) as above but resulting in lack of access to install support systems in sufficient time leading to lining failure (or partial or total collapse due to the lack of it) depending on ground stability.
c) as a) above with failure of the excavation face, requiring an alternative method to resume the drive (unlikely with a full face shielded TBM).
d) As b) & c) above.

6.1.8.3 Fire

In particular with the use of mechanical equipment underground. The use of compressed air exacerbates the fire hazard considerably. If the tunnelling equipment and materials (unlikely with EU or US legislation) are not especially designed or modified for use in compressed air then fire must be considered, either as a primary event or as a secondary consequential event following an earthquake or explosion scenario.

Modern legislation governing the use and storage of hazardous materials underground otherwise mitigates the construction hazard significantly. If hazardous materials or equipment need not be accounted for, then fire is not significant with drives without TBMs, mechanical shields, or roadheaders. However the presence of nearby gas pipes, sewers (especially old ones), or other hazardous underground utilities should always be checked.

6.1.8.4 Explosion

(See also as above regarding nearby gas mains or hazardous utilities.)

This hazard relates in particular to the use and storage of explosives. The maximum quantities taken into the underground works should be checked for the potential damage to the lining or equipment. In the absence of exposures due to nearby utility services as above, this scenario is in Europe, US and Australia, not considered to outweigh alternative PML scenarios.

6.1.8.5 Collapse

Collapse for no apparent reason or otherwise due to an unknown cause, geological anomaly, “mysterious act of God” can affect a limited length of tunnel or the excavation face, either during the drive, or after the lining has been installed. This scenario has been included to reflect a major UK loss from which one can only learn that tunnels cannot be relied upon to remain intact once installed. The TBM and face in this case remained unaffected, but an already completed section of the segmental lining basically “fell apart” due to an as yet unconfirmed cause. There are a number of hypothesis, regarding the hydraulic effects of a coastal tide, wind-blown sands, “organic clay” or peat, incomplete grouting, wood in the ground above the tunnel crown, lack of grouting, segmental gasket failure, or a manifestation all these plus a few other possibilities combined. Whatever the cause the ground over 77,18m to the East, and 39,65m to the West of an intermediate shaft had to be frozen and re-mined.

6.1.8.6 Inadequate design or method of execution: lining failure / collapse

This scenario has particular application with respect the experience and know how of the project stakeholders, engineer, and or contractor, reflecting a major collapse near London, UK.

The NATM philosophy described above was used for the design and execution of the large diameter station profile excavation. As opposed to the scenario given above there were on this project at least 20 causes, any one of which was sufficient to have lead to the total failure of the lining. Lack of monitoring, lack of experience at all levels, deficiencies in the application of the method, etc. Maybe the main reason was that the foreman on one shift decided to remove 20m of invert in need of repair!.

6.2 Bridges & Viaducts

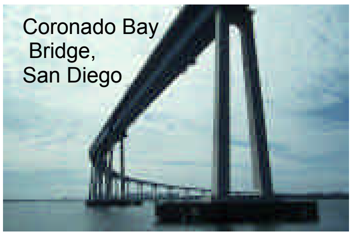
6.2.1 Introduction

The history of bridges and viaducts is directly connected with the development of civilisation and associated transport and communication infrastructure routes. The middle of the 19th century brought the rapid progress of our industrial world with modern traffic systems, accompanied by new demands with bigger loads (dynamic forces) and greater movement frequencies. These capacity demands have promoted many large scale projects, especially for railways needing straight alignments avoiding curves and steep gradients to cater for new generations of very high speed trains.

This has led to sophisticated designs relying on modern material and construction technologies to meet current day transportation infrastructure requirements, and those expected in years to come.

6.2.2 Types of bridges and viaducts – Construction methods

***Girder bridges*.**



Simple bridges as single span girder or multiple-span girders placed over two or more bearings on support columns. The construction materials can be laminated wood, steel and concrete spanning several meters to over 100m.

Frame bridges. Frame bridges are made of steel, reinforced concrete or prestressed concrete. The "frame" is built up by connecting the bridge superstructure (frame transom) to the piles or abutments (vertical members). A typical girder bridge is the Ölands Bridge between the Swedish coast and the island Öland with a span length of 130 m (completed in 1964).

***Arch bridges*.**

This "old fashioned" type of bridge can be well integrated into different topographical landscapes. The main loaded bearing element of this type of bridge is the vault. The arch transfers the loads onto the two abutments, which in turn require good subsoil conditions to provide adequate load-bearing capacity for the whole structure. The most important variations of arch bridges are: solid face arch bridges (concrete), spandrel-braced arch, and bowstring bridge (Tied Arch). A famous steel arch example is the Sydney Harbour Bridge, Australia with a span length of 503m (completed in 1932).

Suspension bridges.

Suspension bridges are often very "slim" structures, principally comprising of pylons, main bearing cables, hangers and bridge decks. The design lends itself well for both relatively short spans to 1.500 m and longer.

Cable stayed bridges. This is a modern type of bridge in use since the beginning of the 1950´s. The materials used for cable stayed bridges are the same as for suspension bridges. The main components are pylons, stay cables and the bridge deck. The Öresund bridge (*shown right*), completed in 2000 between Malmö and Copenhagen (Sweden - Denmark) represents a modern cable bridge with a centre span of 490 m.

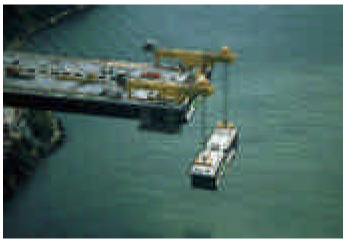
6.2.3 Construction methods

Only a short description can be given here, but the Munich Re publication "Bridges – Technology and Insurance" provides further details with accompanying photographs.

Falsework construction.
Concrete shuttering or form-work for casting in-situ concrete. The formwork for beams and bridge decking is supported by false work which carries the weight of the concrete onto the ground below until the structure is complete, or otherwise until it has sufficiently cured to support its own weight.

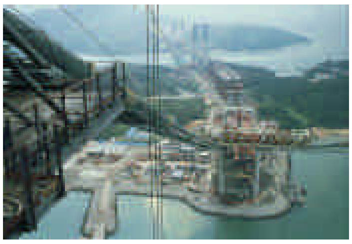
Incremental launching.
The bridge superstructure is fabricated on one side of the bridge in sections, each being cast from stationary formwork erected behind one of the abutments. Once the concrete has sufficiently cured, the section is prestressed, stripped of the formwork and launched out using hydraulic presses on sliding bearings. Subsequent sections are then cast and launched to complete the span between the two abutments.

Precast concrete units.
Pre-cast concrete units (or pre-fabricated steel sections) offers various advantages. The method negates the need for false or formwork required for in-situ concrete. The units are usually brought into position using a launch gantry (mobile lattice girder structure), each connected up with the previous unit to complete the span.



Cantilevering with cast-in-situ concrete.
With this method, the bridge superstructure is produced using cantilevered scaffolding with short sections of formwork. The sections are then joined to the completed portion of the bridge.

Suspension bridges.



The foundations for the towers and the main –cable anchorages are completed first, followed by the towers/pylons from which the cables are hung (*as left*). The bridge deck is then suspended from the cables (*right*).

6.2.4 Hazards and PML scenarios

Technical Segment	**Hazards / Sensitivity Factors ***				
Type of Bridge	Nat. Exp.	External	Collapse	Fire Expl	Construction/Design
Girder	1	1	1	1	1
Frame	1	1	1	1	1
Arch	2	2	3	2	2
Suspension	2	2	2	2	2
Cable stayed	2	2	2	2	2
Components					
Foundations /abutments	1	1	1	1	3
Falsework	1	1	1	1(steel) 3 (wood)	2
Formwork	2	2	3	1(steel) 3 (wood)	2
Bridge deck	1	2	2	1	2
Cantilever	2	2	2	2	2
Pylons	2	2	2	2	2
Main cable	1	2	2	1	3
Caissons	1	1	1	1	2

* Sensitivity factors

0 = unaffected, unlikely to suffer damage

1 = low, minor damage, can be repaired

2= medium, significant damages, may require alternative working method for repair

3 = high, catastrophic failure of bridge, collapse

6.2.5 Catastrophe scenario – collapse/fire

The worst catastrophic scenario is defined as a collapse and/or a false work fire other than with steel only materials. The causes can be due to design and material failures, miscalculations, human errors etc, especially for the foundations which are a critical load bearing component. Certain regions are prone to natural hazards, the severity of which may not have been properly designed for during the intermediate stages of bridge construction, or even for the completed structure. There are thus many sensitive phases which will have a bearing on the appropriate scenario depending on type of bridge, its particular design, the construction methods employed, and geological conditions. The critical factors governing a catastrophic event are:

- *Construction of large span lengths, high bridges*
- *Geological soil conditions*
- *Natural exposures*
- *Sensitivity to windstorm, flood and/or other weather conditions*
- *Complicated lifting procedures (e.g., very large and heavy bridge sections to be lifted and fixed into position)*
- *Traffic during construction – inadequate safety regulations/safeguards (e.g., river boat traffic, open water boat traffic, railways etc)*

- *Design and construction errors*
- *False work failure/fire*
- *Overturning of cranes, launch gantries etc.*

6.2.6 Example 1. PML assessment for a girder bridge construction

Project description: A multi-span concrete girder bridge for a railway double-track over water with water borne traffic during construction.

Technical data: total bridge length: 330m. Ten individual span sections on piled supports, each section has a length of 33m

Construction period: approximately 30 months.

PML scenario: impact by a water vessel (large freight ship) to piled supports for the bridge deck during the most critical phase, i.e. when the supports are not protected and the total static load of the bridge structure has not been stabilised.

PML Calculation:

Construction element	PML, % damages to permanent works
1. Piles	20
2. Debris Removal	100
3. Bridge deck (surface)	20
4. Bridge girders	20

Other estimated possible PML scenarios:

Flood: statistics from the weather authorities indicates water level variations of + 69 cm maximum over normal mean water level within the last 50 years. The flood risk is estimated as low.

Earthquake: Earthquake Zone = 0 (Munich Re map of natural hazards). The risk is nil.

Aircraft impact: No commercial flight paths proximate to the site servicing the nearest main airport. The risk is insignificant.

Sabotage, burglary: Sabotage damage to the works is a plausible risk. The site (land side works) is usually secured by a perimeter fence, which typically for civil engineering construction sites is not very secure and it's always possible to access the site by boat. However experience from insuring such risks has demonstrated that heavy civil projects are not targeted for sabotage, the risk will more likely lead to premium attrition than a PML event.

Motor vehicle traffic: The site has no direct connection (road closed) to public traffic. Potential damage caused by road (or site) vehicles is considered only very limited, and is estimated as low.

6.2.7 Example 2. PML for a suspension bridge

Project description: A modern suspension bridge made of reinforced concrete crossing a main river. Ships (freight ship traffic) are passing the site during the construction period.

Technical data: built in 1997. The free height over the river is 40m. The pylons tower 189m above river level. The span length is 1.210m.

Construction period: 20 months (+ 2 years maintenance period)

PML scenario: The PML scenario is based on a storm occurring during a critical stage of erection before the bridge sections have been totally joined together. The bridge deck starts to swing and subsequently collapses.

PML Calculation

Construction element	PML % damage to permanent works
1. Debris Removal	100
2. Extra costs	100
3. Steel/concrete works	67
4. Hanging system	10

Other estimated possible PML scenarios:

Flood: The flood risk is estimated as: zero/low.

Earthquake: The site is located in earthquake zone = 0 (Munich Re map of natural hazards). The risk is estimated as low.

Aircraft impact: No flight paths proximate to the site. An aircraft impact on one “standalone” pylon before completion of the bridge construction may cause a significant loss. This scenario is normally not within the PML definition.

Sabotage, burglary: Sabotage damage to the works is a plausible risk. The site (land side works) is usually secured by a perimeter fence, which typically for civil engineering construction sites is not very secure and it’s always possible to access the site by boat. However experience from insuring such risks has demonstrated that heavy civil projects are not targeted for sabotage, the risk more likely leading to premium attrition than a PML event.

Motor vehicle traffic: The site has no direct connection (road closed) to public traffic. Potential damage caused by road (or site) vehicles is considered only very limited, and is estimated as low.

6.3 Pipelines / Trenches

The following relates only to the civil works associated with pipe laying projects and does not deal with the material hazards relating to the pipeline itself (e.g., deformation, corrosion, welding, etc).

6.3.1 Construction methods

6.3.1.1 Pipe laying on land

The civil works required for pipe laying projects are mainly earthworks. However specific structures such as bridges, tunnels, underpasses, anchor blocks, may also be required depending on the topography and possible manmade obstructions (e.g., highways etc.) along the proposed alignment. Such additional structures that often accompany a large pipeline project are outside the scope of this section (but are otherwise dealt with in other sections of this paper).

Pipelines are most commonly buried in a trench prepared with a sand bed to provide an elastic mattress underneath the pipe. The minimum earth cover above the pipe is usually 1m except when the pipeline passes under roads or rivers (see below).

The works programme must be planned carefully so the various operations can be carried out in the optimum sequence without interruption, keeping the open trench time to a minimum.

Intermediary stockpile depots of pipes are placed along the alignment to aid the logistical supply of permanent materials for the pipe laying operations

Pipe elements are then welded together and the whole section is laid in the trench which is subsequently backfilled.

The length of open trench prepared for a pre-welded length of pipe is, depending on local conditions, unavoidably vulnerable to flood and inundation perils. It's generally not commercially viable to protect all the open trenches with major pipeline against flood events. Such measures may even cost more than the trench itself, or else aggravate the overall exposure due to delays in the programme. Protection over limited lengths would however be appropriate for very deep sections.

A pipeline may otherwise need to be secured against buoyancy or movement in flood prone regions using soil or rock bolts.

6.3.1.2 Pipe laying in water

Siphons:

Pipeline projects can involve river crossings of various types and sizes, from small water courses or canals to rivers several hundreds of meters wide. Siphons require specific methods of working and construction equipment (cranes, pontoons) according to the width of the crossing.

If the material carried/transported in the pipe is less dense ,or lighter than, water, the pipe elements are coated with concrete to counteract buoyancy problems.

For canal crossings or narrow rivers, the pipeline section is simply laid in a pre-excavated trench across the river bed. The pipe is subsequently backfilled and can in addition be protected by blocks of rock.

Wide river crossings involve several cranes, possibly pontoon mounted units to carry the pipeline over the whole width of the river. The pipeline is then laid in the trench simultaneously by all cranes in tandem. Since one unit alone cannot support the weight of the pre-welded pipe section this operation is difficult to ensure none of the units become overloaded and topple or capsize into the river.

An alternative method is to fit the pipeline with floaters and to pull the whole section length spanning the width of the river from the opposite bank, and to sink it slowly by filling the floaters with water.

<u>*Offshore pipe laying:*</u>

Offshore pipe laying is carried out from a pontoon equipped with all facilities necessary for welding, monitoring etc.

The pontoons are designed with a ramp to allow the pipeline to slide down into the sea. The anchor point of the pontoon moves forward step by step along the pipeline alignment as work progresses.

This 'standard' method can be used for depths of up to approximately 100 m whereby the progress rate under normal conditions is about 1.5 – 2 km per day. For depths over 100 m., other methods have been developed which fall outside the scope of this paper.

It is often necessary to bury the pipeline in a trench excavated in the sea bottom to protect it against hazards such as ship anchors or scouring erosion caused by tides and currents.

In coastal areas, the pipeline is usually buried at 2 – 4 m depth. The cost of such works can be significant and may represent up to 25% of the total project value.

If the open trench is likely to be exposed to storms and undercurrents, its length should be kept to a minimum. The transition zone between deep waters and dry land is in this respect particularly exposed.

6.3.1.3 Trenchless works

<u>*Pipe-ramming:*</u>

Pipe-ramming allows road crossings to be carried out without any interference to the traffic. It is usually performed for horizontal installations and limited to relatively short lengths (up to 40m).

It consists of driving the pipe into the ground using hydraulic jacks or compressed air. A cutting tool is fixed at the end of the pipe and the excavated material is removed mechanically, hydraulically or manually.

Directional drilling:

This method is now widely used, in particular for river crossings to reduce the flood related risks.

The works are carried out in two basic steps: a rotating drilling head is used to carry out a "pilot" hole (~50mm diameter) whilst bentonite is constantly pumped down the drill string (attaching to the drill bit) to remove the drill cuttings, and to support the excavated surface of the hole. A guiding device fixed to the drilling head provides information on its position to facilitate the correct alignment.

Once the drilling head reaches the surface at the other side of the river, it is replaced by a reamer to complete the hole to the final diameter, possibly in several passes. The pipe is simultaneously installed behind the reamer bit (with the final pass if more than one is required), acting also as a drill casing to prevent collapse of the hole directly behind the reamer bit.

6.3.2 Hazards and PML scenarios

The main risks associated with pipeline construction are:

Flood, depending on the location of the project and the natural perils exposure.

Inundation and heavy rain, storms, typhoons and hurricanes can lead to huge losses. The risk is particularly high in deserts or sand-covered regions where storms occur rarely, but when they do very suddenly and with high intensity. Very dry conditions often produce an impermeable ground surface that aggravates the effects of flash floods leading to a total loss of open trenches and excavation pits.

- *Specific loss protection measures must be taken to protect the site against flooding, the most important being to ensure the open length of trench is kept to a minimum for as short time as possible.*
- *The diminished flood exposure to the works after a trench has been back-filled means the PML estimate should be based on the maximum length of open trench during the project, assuming a total loss of completed civil works for the section length identified (or otherwise equivalent to the policy limit for open trenches).*
- *The same applies to trenches excavated under water as above.*

Landslide: To be considered if the works are carried out in unstable areas prone to landslides, or indeed if the earthworks themselves may potentially trigger such an occurrence. Landslide events can affect completed sections of the pipeline (but otherwise not handed over), as well as sections still under construction.

For the trenchless method, collapse (in addition to possible material damage to the pipe itself, which is not discussed here) is the main loss scenario and can affect 100% of the section potentially affected.

The above is summarised in the following table:

Construction method	Hazards / sensitivity factors		
	Flood/Storm	Landslide	Collapse
Conventional	3	2	1
Trench-less	-	-	2

1 = minor or limited damage ; 3 Total collapse

6.4 Roads / Railways

This section does not include any other project related structures such as buildings, bridges etc. Both Roads and Railways are exposed to a variety of perils that by and large depend on the following factors:

- *Geographical and Geological Situation*
- *Type of Road or Railway*
- *Design and Construction Methods*
- *Experience of project Stakeholders*

6.4.1 Geographical and Geological Situation

- *Coastal Area*
- *Desert Zones*
- *Midland / Country - Area*
- *Urban / City -Area*
- *Hilly Area*
- *Mountain Area*
- *Condition of the soil*
- *Soil with a high likelihood of subsidence and landslide*
- *Slope and embankments*

6.4.2 Types of Road and Railways

- ➢ High-speed railway tracks and roads (motorways)
- ➢ Normal speed railway tracks and roads (e.g., local connections only)
- ➢ Trams and similar tracks and urban and City roads
- ➢ Road and rail routs in mountainous terrain

6.4.3 Design and Construction Methods

A Road with base course, with covering layer of cement or bitumen
B Road with base course, covered with cobbled pavement
C Roads and runways without base course
D Special Railway tracks like those for magnetic - and suspension - railways
E Rails with base course
F Rails on Rock or concrete body

6.4.4 Hazards

The main exposures for road / rail construction projects arise from:

- Earthquake
- Typhoon, windstorm, tsunami
- Flood, inundation, proximity to watercourses, lakes
- Landslides, mudflows, rock falls in mountain areas
- Frost and heat

6.4.5 Experience of Project Stakeholders

The quality and experience of the stakeholders involved in the project, including the engineers, consultants, project management, site management, contractors etc. play a critical role besides and the use and extent of mechanical equipment, the construction methods employed, and the topography to be negotiated.

6.4.6 PML Scenarios

possible damage to:

- *excavation and embankment works, the coffer or the covering layer caused by landslide , rock fall, rain, earthquake, storm, flood, inundation, depression of embankments and coffers caused by waves in coastal areas*
- *bitumen- asphalt - or cement- covers caused by fire*
- *road installations such as the ornamentals, the illumination, signalling, guiding rails, traffic lights.*
- *sleepers and the rails caused by fire whereas the welded rails may suffer enormous (differential expansion) damage whilst being exposed to heat.*
- *railway installations such as the cable trays, the bus bars, the conductors, signalling, etc.*
- *new technology used for D systems can especially result in heavy damage to the expensive electromechanical equipment built in the tracks and civil works components.*

The PML will be based on the most exposed length of the insured alignment that would constitute a total loss, according to cost of construction per meter multiplied by the length of the defined sector.

In addition – according to the scope of cover - additional cost factors must be taken into account:

- existing property / buildings
- replacement or exchange of earth and soil (slopes and embankments)
- additional cost for maintaining existing traffic facilities
- additional cost for construction equipment and machinery

- additional cost for balance of plant and camp facilities
- additional costs for stores and stored material

6.4.7 PML Matrix

Technical Segment	Hazards / Sensitivity factors *			
	Quake	Flood	Boulder-blow	Fire
A	1	3	1	1
B	2	2	2	1
C	2	3	2	2
D	3	3	2	3
E	3	3	2	1
F	2	2	2	2

** Sensitivity Factors*
1 = low
2 = medium
3 = high

6.5 Dams and related Hydraulic Structures

6.5.1 Main types of dams

6.5.1.1 Gravity dams

Gravity dams rely solely upon their weight for stability. They must be secure against overturning and sliding at any horizontal plane within the dam and at the plane of contact with the foundation, and they must be designed in such a way that allowable stresses in both the dam and the foundations are not exceeded. The two main types are Roller Compacted Concrete and Embankment Dams, detailed further on in this section.

Loads acting on the dam include the following :

Horizontal loading

- *Hydrostatic pressure on the upstream face*
- *Ice load against the upstream face*
- *Dynamic load of water against the dam due to earthquake*
- *Inertia force of the mass of the dam due to earthquake.*

Vertical loading

- *Weight of the dam*
- *Uplift force on any horizontal plane within and under the dam*
- *Inertia force of the mass of the dam due to earthquake.*

Various devices can be used to reduce the uplift forces (which have caused several dam failures in the past) including:

- *Grout curtain at the heel of the dam*
- *Drainage holes downstream of the grout curtain*

Roller Compacted Dams

Roller compacted concrete (RCC) is handled like earthfill allowing lower cement content and therefore lower cost. However, appropriate account must be taken of the following two aspects:

- *Mechanical strength, in particular shear strength of horizontal planes between concrete layers (usually 30 to 60 cm thick), due to the smooth surface induced by the compacting method.*
- *Sealing of the dam which is usually achieved by a concrete face or a membrane.*

Embankment dams

Embankment dams mainly include rockfill and earthfill dams. Earthfill dams can be either zoned - involving selected areas of rock, gravel, earth and impervious zones - or homogeneous. They do not need to be built on rock, and are not vulnerable to "soft" foundations such as decomposed rock or alluvial deposits.

However they are quite sensitive to overtopping, which can very rapidly lead to dam failure caused by superficial and internal erosion of the downstream fill. For this reason, spillways associated with earthfill dams must properly cater for this problem, especially in areas where long term historical river flow data is not available.

The stability and safety of an embankment dam relies on two key design parameters :

- *Control of the pore water pressure within the embankment, including during construction*
- *Control of seepage within the embankment to prevent internal erosion (piping)*

A full stability analysis considers the following conditions:

- *Normal conditions: reservoir full;*
- *End of construction: reservoir empty. Pore water pressure is high as pressures resulting from construction are not yet dissipated;*
- *Rapid drawdown: after drawdown, pore water pressure due to the reservoir is not relieved, and tends to destabilise the upstream fill of the dam;*
- *Seismic conditions, including dynamic load from the reservoir and inertia forces.*

In order to control and reduce pore water pressure within the embankment, internal drains are provided, in particular:

- *a vertical drain downstream of the core (or in the central part of an homogeneous dam);*
- *a pervious blanket, over the downstream half of the foundation which conveys seepage water to the downstream toe of the dam.*

In addition, piezometric cells are installed within the embankment for monitoring purposes.

Internal erosion is avoided by means of filters provided between low grain-size zones and drains, preventing fine material from being carried away by seepage water.

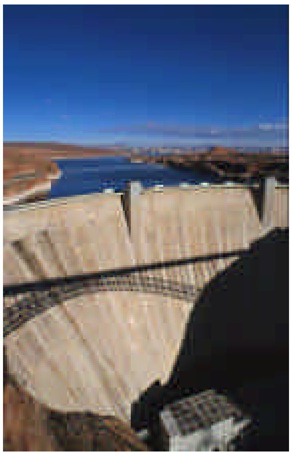
This phenomenon, known as “piping”, severely jeopardises the integrity of the dam as it accelerates rapidly once started.

Rockfill dams are less sensitive to pore water pressures due to the high permeability of fill material. They permit steeper slopes of the dam face, but as a consequence the loads on the foundations are higher.

Water tightness of rockfill dams can be achieved by

- *Internal core (e.g., clay)*
- *Concrete facing (either cement or bituminous concrete)*

6.5.1.2 Arch dams



Arch dams are based on the principle that the load is transferred to the abutments by an arch structure with a geometry ensuring that the materials used are only in compression. Concrete can better resist compression loads than tension, ensuring a slender or efficient design with less volumes of concrete than would otherwise be required for a straight wall for example. Various geometrical designs can be employed from a vertical cylindrical upstream face with a uniformly inclined downstream face to the double-curvature type.

The latter is often the preferred dam type for heights over 150 m and can be considered very safe: the only known failure (Malpasset in France, 1959) was not due to the arch itself, but rather to the failure of the foundation under very unusual geological and pore water pressure conditions at the site location.

One key criterion for selection of an arch dam site is the low deformability of the foundation rock which varies according to the number of discontinuities (cracks, joints, faults) and the geo-mechanical properties of the rock material itself. It is essential that all parameters are identified, requiring detailed and comprehensive geotechnical investigations for optimum site selection.

In addition to the thrust of the dam acting upon the abutments, the forces induced by the pore water pressure from the reservoir and acting within the foundation can also prove critical. For this reason, the abutments of arch dams should always be equipped with effective, well maintained and monitored drainage systems.

6.5.2 Other hydraulic structures

6.5.2.1 Spillways

The role of a spillway is to release a flood (or that part of a flood in excess of the volume stored in the reservoir) downstream of the dam. It is commonly designed for a return period of 10 000 years or for the probable maximum flood (PMF) obtained by statistical analysis of historical river flow data. Concrete dams are less exposed to overtopping failure and can be designed for shorter return periods.

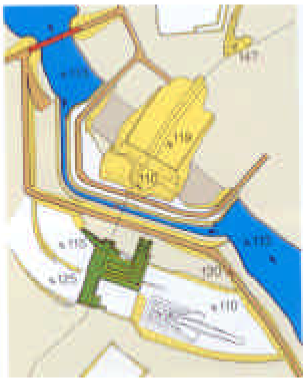
Spillways for concrete dams are incorporated in the dam face, comprising gated or un-gated sills (overflow dams) with a flip bucket or a stilling basin at the downstream end to dissipate the energy of the nappe. These structures often involve the construction of large concrete slabs, for which adequate foundation drainage should be provided to prevent uplift.

Spillways associated with embankment dams are independent or “stand alone” concrete channel structures located on the slopes of the river valley.

The flow velocities in steep sided spillways or chutes can reach 30 m/sec or more. The cavitation effects generated with such conditions are prevented with aeration devices to avoid erosion of the concrete surface.

Another type of spillway, known as “morning glory”, consists of an intake tower located in the reservoir connecting to a tunnel which releases the flood downstream of the dam.

6.5.2.2 Diversion works



Dam projects invariably involve major river diversion works to provide dry access to the site and river bottom for the foundations. The construction site is usually protected by cofferdams (embankments or cellular cofferdams) and the diverted volumes of water are re-routed in tunnels or open channels in wider valleys.

Construction of the dam is usually carried out in several consecutive phases, each of which corresponds to high or low flows with different discharge requirements.

Narrow valleys invariably call for a diversion tunnel associated with upstream and downstream cofferdams. The capacity of the tunnel depends on the hydraulic head at the intake, which will in turn be a function of the height of the coffer dam.

The design flood return period of the diversion works (sometimes called “the construction flood”) usually varies between 10 and 100 years. However for earthfill dams, the return period should be at least 20 – 50 years (sometimes more) depending on design, given the catastrophic consequences of a flood event.

The main risks associated with diversion works are :

- *Failure of the upstream (or downstream) cofferdam due to faulty design or workmanship*
- *Overtopping of the upstream (and/or downstream) cofferdams due either to a significant flood (higher than the construction flood) or to the inadequate design of the diversion system.*

Both events would result in the partial or total destruction of the cofferdam(s) and significant damage to the downstream construction works.

6.5.3 Hazards and PML scenarios

6.5.3.1 Hazards / Sensitivity factors

The following main hazards have been considered with severity factors of 1 to 3, equivalent to minor consequences (1) to catastrophic failure (3):

Type of dam	**Hazards / Sensitivity factors**				
	Earthquake	Flood / Overtopping	Uplift	Piping	Foundation Failure
Roller Compacted Concrete	1	1	2 ✶	-	-
Embankment dams	1	3	-	3 ✶	-
Arch dams	2	1	1 ✶	-	3 ✶
Specific structures / works					
Spillways	1	-	2✶	-	-
Diversion works (cofferdams)	2	3	-	2	1

✶ *Mainly after or during first filling of reservoir.*

6.5.3.2 PML scenario

A dam project usually includes several components. Depending on the purpose, these can be :

- *the dam itself, the intake structure, the spillway,*
- *the headrace channel or tunnel,*
- *the powerhouse or water supply facilities,*
- *the access roads etc.*

Each component must be considered separately and the worst case must be selected as the scenario for the whole project. The selected scenario – for example a flood – can however affect several project components.

The ICOLD (International Commission on Large Dams) has published statistics on accidents which have affected large dams. These statistics have demonstrated that :

- *the probability of failure of a dam irrespective of its type, age or location is about* 2.10^{-5} *per year (1: 50 000).*
- *The risk of dam failure is the highest during the first filling of the reservoir : 2/3 of the accidents occur during this period for embankment dams, and ½ for concrete dams.*
- *Most failures (75 %) affecting embankment dams are caused by surface or internal erosion, i.e. flood or piping.*
- *Concrete dams are two times safer than embankment dams. Most accidents (75%) affecting concrete dams are due to a failure of the foundation. There are usually signs preceding a failure (cracks etc) which allow emergency measures to be taken.*

It can be concluded from the above statistics that the most appropriate PML scenarios are:

- *Embankment dams: flood during construction or internal erosion during first filling of reservoir, resulting in dam failure with 100 % destruction (except in wide valleys where the failure can be limited to a portion of the total length).*

- *Concrete dams: Concrete dams: earthquake or foundation failure during the first filling of the reservoir resulting in extensive damage to the dam. A percentage of the dam value (25 to 50%) is usually taken as an estimate of the necessary repair works (grouting etc). A possible maximum loss should, however, be 100% of the value of the dam.*

An MFL scenario (rather than a PML scenario) would be a total loss (100%) of a dam in either case.

Major river development schemes involve a series of dams one downstream from the other that are dependent on an overall water management programme. The operational water requirements of each rely on the release of water from the upstream dams. Accidental releases can cause a chain reaction downstream, and potentially overtopping or flooding problems if not monitored carefully.

6.6 Harbour Works

6.6.1 Introduction

Both sea and inland harbours were the first trade and transportation places for connecting countries and continents as well being of strategic military importance. They are still home to vast commercial and military vessels, and many harbours world wide have grown from the small fisher-boat quays and trading docks to fully integrated infrastructure schemes with harbours and terminals for passengers and goods. Many are equipped with sophisticated high volume container handling facilities, as well providing specific arrangements for servicing the chemical, gas and oil industries etc. Despite their size and modern designs the principle harbour components, including piers, breakwaters and jetties are still very vulnerable to the forces from windstorm and sea waves both during construction and after.

6.6.2 Types of harbours – Construction methods

This section only considers the risks for "conventional" coastal harbours or those within larger inland waterways. It does not take account of shipyards, special harbours and terminals for offshore gas and oil installations.

Most modern harbours use "heavy" materials, mainly steel, concrete, rock stones and combinations of these products, in their construction to resist against the extreme forces from water and windstorms.

<u>The most relevant aspects to be considered for a PML are:</u>

Site plan provides information on the tidal / coastline exposure, sea wave (fetch) protection, water traffic, roads, railways etc.

Foundation conditions

The foundation designs must be based on expert reports for the local geological conditions, taking account of drilled samples and borehole logs. The conditions in combination with the effects of water and wind storms are of vital importance in delineating the design parameters and selecting of the appropriate construction methods.

Sophisticated and expensive foundations are often necessary to cater for the large loads /surcharges that have to be supported by the quay structures, exacerbated by the storm resistance requirements and ship impact loads. Whereas on land well foundations or pile foundations are used, under water foundations require complicated structures with caisson methods of installation.

6.6.3 External influences

Weather data is of vital importance especially regarding floods, tides, windstorms, rain, frost, ice, temperature variations, etc. Earthquake data, and the proximity of nearby faults is of equal importance in active areas. All this data must be collected and analysed prior to construction works.

6.6.4 Hazards and PML scenarios

Technical Segment	**Hazards/Sensitivity Factors ***				
Harbour Types	Nat. Exposure	External	Collapse	Fire/Expl	Construction/Design
Quays	2	2	2	0	2
Breakwaters	3	3	3	0	2
Jetties	3	3	3	1	2
Construction Segment					
"Open" sections	3	3	3	1	3
Foundations	2	2	2	0	2
Caissons	2	3	3	0	2
False works	2	2	3	3	2
Piling	2	2	2	0	2
Supports	2	2	2	1	2
Overturning	2	2	3	1	1

* Sensitivity factors

0 = unaffected, unlikely to suffer damage

1 = low, minor damage, can be repaired

2 = medium, significant damages, may require alternative working method for repair

3 = high, catastrophic failure of construction, collapse

Catastrophe scenario – collapse

The worst catastrophic scenario is defined as collapse into water. The causes can be design and material failures, storm water and wind forces, miscalculations, human error etc. In certain areas the natural hazards will often be of paramount consideration. A PML calculation should be based on information from weather authorities and also authorities for design and construction of harbour and coastal structures.

A lot of critical phases will depend on the construction methods employed for the quays, breakwaters, jetties, as well as the prevailing natural and geological conditions. The critical criteria /construction phases on which a catastrophic collapse scenario is usually dependant are:

- *Construction of large "open" sections of quays, breakwaters or jetties*
- *Geological soil conditions*
- *Natural exposures*
- *Sensitivity to windstorm, water waves, flood and/or other weather conditions*
- *Complicated concrete structures to be completed off-shore, lifting procedures etc.*
- *Water traffic during construction – inadequate safety regulations/safeguards, safety distance etc.*
- *Design and construction errors*
- *False and formwork failures*
- *Overturning of cranes, construction machines, concrete lorries etc.*

6.6.5 Examples

PML Assessment

Project description: Construction/reconstruction of a breakwater with a length of approximately 850m for a harbour project in the Baltic Sea. The breakwater is exposed to the South, Southwest and West to storm waves generated over a fetch of up to approximately 500 km. The design wave height is 4.8 meter (return period 50 years).

Technical data: The breakwater comprises of approximately 75.000 tons of heavy stones (< 12 tons each).

Construction period: 7 months

PML Scenario: impact by a combination of storm and water wave forces acting on the armour stone blocks protecting the breakwater during the most critical phase, i.e. where the stone blocks are not completed with filter stones etc.

PML Calculation

Value of breakwater equivalent to a "open" length of 200 meter as foreseen in the construction programme (otherwise per indemnified unprotected length in the policy).

Other estimated possible PML scenarios:

Earthquake: Earthquake zone = 0 (Munich Re map of natural hazards).

Environmental risks: The risk is estimated as low.

Waterborne vessels: Damage due to waterborne vessels is possible. The damage is calculated to be less than the influence from storm/water waves in the above PML Calculation.

Aircraft impact: No flight path to the nearest main airport exists proximate to the site.

Sabotage: Damage due to sabotage cannot be neglected. The site is not normally fenced in, but access to the site from the water exists. The experience from insuring similar risks has demonstrated that the sabotage risk for this type of project is low.

Motor vehicle traffic: No risk.

7 UNDERWRITING CONSIDERATIONS

7.1 Minimum PML

It is usual practice within companies to lay down a minimum PML quantum expressed as a percentage of the total sum insured to account for the uncertainties of PML estimation. One minimum PML for the whole CAR business branch is, however, considered appropriate only if it reflects the sensitivity of the technical risk actually at stake. Ideally such PML Guidelines should be broken down into the major technical segments as indicated under Section 6 of this paper.

7.2 Policy Cover

The wording must always be applied to material damage PMLs to assess the ultimate exposure under the policy. This may well be restricted by sub- or policy limits, first loss limits, or exacerbated by extensions. Special covers need in particular to be evaluated for the chosen scenario.

The following list is given for the most common extensions which should be loaded to the base PML:

- expediting expenses
- removal of debris
- escalation / indexation clauses
- plant and equipment
- existing property
- architects and engineer fees
- custom duties
- increase cost of working

7.3 PML Format

A PML should be documented such that it can be reproduced by another risk engineer or underwriter, especially if it needs to be checked in lieu of additional information (maybe after a risk survey) or a change in a projects' circumstances (extensions etc). The most appropriate way of ensuring a consistent format necessary to cover all eventualities is by means of a Standard PML Evaluation document, that should contain interalia all loadings to the base PML as above. This will also enable the "PML history" to be followed, or if necessary to trigger standard rechecks for insurance periods greater than 2-5 years.

8 CONCLUSIONS & RECOMMENDATIONS

The main conclusions of PML estimation are:

- *It is not an exact science*
- *There is no common standard*
- *Insurance companies employ a variety of Definitions, Methods, and Philosophies.*

The approach is subjective and open for different interpretations and these interpretations can be costly.

The object of this paper is to assist in PML evaluation for civil engineering projects, and in particular in securing an accurate and precise assessment.

It is also important to remember that a major claim is rarely contingent on only the Estimated Final Construction costs or original construction costs, which may vary according to the construction contract, but also on the possibly unique circumstances of the loss scenario. Engineering projects can seldom be completed out of sequence, and likewise repair costs may rocket over the original construction costs, in particular with tunnelling risks for example, or due to access problems, remobilization, additional temporary works etc.

It is essential to determine the PML according to sound underwriting data and – wherever possible – by involving expert engineers particularly for large industrial risks. A PML may not reflect any kind of commercial interests an underwriter may have, and consequently once a PML is determined it should not be "over-ruled" or compromised for any reason lacking a sound technical basis.